U.PORTO
FEUP FACULDADE DE ENGENHARIA
UNIVERSIDADE DO PORTO

CONCEPTION DESIGN AND STRUCTURAL ANALYSIS OF MOVABLE BRIDGES

The Case Study of Great Yarmouth Third Crossing

ANA BEATRIZ ESTEVES RAMOS

Dissertação submetida para satisfação parcial dos requisitos do grau de

MESTRE EM ENGENHARIA CIVIL — ESPECIALIZAÇÃO EM ESTRUTURAS

Orientador: Professor Doutor Humberto Salazar Amorim Varum

Coorientador: Doutor Pedro Gonçalo Faustino Marques

JULHO DE 2017

MESTRADO INTEGRADO EM ENGENHARIA CIVIL 2016/2017

DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440

✉ feup@fe.up.pt

http://www.fe.up.pt

To my mother and brothers,

“Every Master was once a beginner.

Every pro started as an amateur.

Every icon began as an unknown.”

Robin Sharma

ACKNOWLEDGMENTS

In first place I would like to thank my mother for all the unconditional support she has given me throughout my life and especially in college years. To my brothers for being my best friends and supporting me when I need.

To Professor Humberto Varum for making this international experience possible and with it not only improve and develop my professional knowledge but also allowing me to have the best personal experience that I have ever had.

To Engineer Pedro Marques for encouraging me throughout the all process of my dissertation and internship. His dedication, guidance and feedback were essential for my good performance and I thank you for his willingness to review all my ideas and documents.

To my co-workers at Mouchel/WSP, who have provided the best working environment and helped me through all the problems I encountered throughout my internship. I would like to particularly acknowledge Sam Akerstrom and Aram Hassan for all the guidance and support given throughout the project of GYTC.

To my “sisters” Mariana and Claudia for their friendship and care. I could not have asked for better friends.

To Marta, that was one of my main supports in my Erasmus journey. This experience gave me more than professional and personal growth, gave me a friend for life.

To my friends for being with me in the best and worst moments of my life, with them I could go through the challenges of life with a smile on my face.

RESUMO

Pontes móveis começaram a ser construídas desde tempos primórdios e numerosas pontes de diferentes tipos e materiais foram construídos até o presente. Com a evolução dos materiais e dimensões dos navios, o que, consequentemente, trouxe maior tráfego de navegação, o design de pontes móveis teve que melhorar constantemente.

Ao contrário das pontes fixas, as pontes móveis são uma combinação de dois designs técnicos, estruturais e mecânicos/elétricos/hidráulicos que devem estar interligados ao longo de todo o processo de design, requerendo, portanto, uma ampla gama de conhecimento e pessoas envolvidas.

O objetivo desta dissertação é descrever a metodologia e os principais aspetos que exigem atenção no início da conceção de qualquer projeto de pontes móveis. Além disso, também se descreve alguns problemas que foram analisados com o intuito de perceber como abordá-las à medida que o projeto progride.

Mostra-se que uma decisão sobre o tipo de ponte é muitas vezes governada pelo desempenho operacional da ponte, considerando a navegação e o tráfego rodoviário, e não da própria conceção estrutural da ponte.

No final, foi possível realizar um projeto conceptual de um caso de estudo real, realizado em ambiente empresarial na empresa Mouchel Consulting/WSP e selecionar o melhor design em relação a todos os requisitos e restrições impostas. Por conseguinte, foi estabelecido que uma ponte móvel basculante era o esquema recomendado e foram realizados cálculos preliminares para atestar a sua viabilidade e os desenvolvimentos futuros para a possível construção.

Durante a fase de projeto preliminar do caso de estudo, foi investido muito tempo em aspetos estruturais importantes, tais como: o tipo de bloqueio e o seu efeito sobre a articulação entre as duas folhas; e a presença e localização dos aparelhos de apoio para cargas variáveis, com impacto direto na distribuição de cargas e reações nos suportes. Estes dois aspetos foram identificados como os mais críticos para design de pontes basculantes e uma discussão foi dedicada ao seu impacto no comportamento estrutural deste tipo de pontes. Conclui-se que, ainda é necessário alguma discussão em futuras abordagens do projeto.

PALAVRAS-CHAVE: móveis, ponte, design, conceção, análise estrutural

ABSTRACT

Movable bridges started to be built ever since early times and numerous bridges of different types and materials were built up until now. With the evolution of materials and development of ships dimensions, which consequently brought higher navigation traffic, the design of movable bridges had to constantly improve.

Contrarily to fixed bridges, movable bridges are a combination of two technical designs, structural and mechanical/electrical/hydraulic design which have to be interconnected throughout the whole project design process, having therefore a wider range of knowledge and professionals involved.

The objective of this dissertation is to describe the methodology and key aspects that require attention at the early conceptual stage of movable bridge designs followed by preliminary design. In addition, different issues were also analysed to understand how to address these as the project progresses.

It is shown that a decision regarding the type of bridge is often governed by operational performance of the bridge considering navigation and road traffic rather than the structural conception of the bridge itself.

In the end, it is possible to undertake a conceptual design of a real case study, carried out in the company Mouchel Consulting/WSP and select the best design regarding the all the requirements and restrains imposed. It was therefore established that a bascule movable bridge is the recommended design and preliminary calculations were carried out to attest the feasibility and future developments for possible construction.

During the preliminary design stage of the case study much time was invested on important structural aspects such as: the type of lock and its effect on the articulation between the two leaves; and the presence and location of the live load bearings with direct impact on loads distribution and support reactions. These two aspects were identified as critical plus the design of bascule bridges and significant discussion was dedicated to their impact on the structural behaviour of this type of bridges. It is concluded that further discussions are still required in future design approaches.

KEYWORDS: movable, design, conception, bridges, structural analysis

TABLE OF CONTENTS

INDEX OF FIGURES

INDEX OF TABLES

SYMBOLS, ACRONYMS AND ABBREVIATIONS

k – stiffness

s_k - characteristic ground snow load

f_{ck} - Characteristic compressive strength

fy – Yield strength of steel

Ψ – Factor applied to accompanying action

γ - Partial factor (applied to action)

MEd – Applied bending moment

MRd – Bending moment capacity

Ψ^*_{sv} – Reserve factor (BD 101/11)

V_{Ed} – Applied shear

V_{Rd} –Shear capacity

WL – Wheel loading

δ – displacement

Ni – Axial load on an individual pile

N – vertical load on the pile group

Mx – Bending moment in x direction

My – Bending moment in y direction

Q_s – compressive resistance

α – factor related to bored surface

Cu – undrained shear strength

As – Area of pile surface

Q_b – base resistance

q_b – characteristic values of base resistance

A_b – Area of pile base

Q_T – compressive resistance of a pile

Fs – Safety factor

p – load

l – length

E – Modulus of deformability [MPa]

I – Inertia

Ka – coeficiente de impulso ativo

n – number of piles

N_{SPT} – number of blows in the standard penetration test

AASHTO – American Association of State Highway and Transportation Officials

AREMA – American Railway Engineering and Maintenance-of-Way Association

NEN - Nederlands Normaisatie Instituut

DIN - Deutsches Institute fur Normung

GYTRC – Great Yarmouth Third River Crossing

LEP - Local Enterprise Partnership

UK – United Kingdom

M&E – Mechanical & Electrical

FRP - fibre-reinforced polymers

LM – Load model

TS – Tandem System

UDL – Uniformed Distributed Load

SV – Special vehicle

STGO – Special Types General Order

EN – Eurocode

NA – National Annex

ULS – Ultimate limit state

SLS – Service limit state

DL – Dead loads

EQU – equilibrium

STR – structure

GEO – geotechnical

2D – 2 dimensions

3D – 3 dimensions

1

INTRODUCTION

1.1. PREFACE

With the construction evolution and consequent roads and railways progressive development, emerged the need for the population to make crossings, not only to overcome geographical obstacles, but also to reduce the necessary travel time. However, one of the major problems until today has been the crossing of navigable waters. As one of the key means of global transport, sea-river navigation could not be less important than the construction of bridges, which would affect its course. It was then necessary to find a solution that would enable navigation of rivers and at the same time their crossing. Since building bridges with adequate clearance for the passage of ships required a more elaborate work in terms of very high inclination, movable bridges became the most viable solution to this dilemma, despite all the challenges on their design.

The term movable, refers to the type of bridge that changes position, vertically or horizontally, enabling then the passage of boats/ships into rivers and/or navigable channels when necessary.

Nevertheless, movable bridges have not always had this purpose. In the Middle Age, these were used for protection against enemy's armies. The so-called drawbridges in medieval castles were designed onto the surroundings of castles so that, with or without counterweights, these were lifted through the upper rotation of sheaves, operated with chains or ropes. These bridges, after elevation, were intended to serve as shield against invasions and entry obstruction.

Since ancient times, this type of bridges has undergone a great evolution, not only in its purpose, but also in its techniques and construction design, due to the development of a wide number of technologies and construction materials.

The design of movable bridges, have primary explored over the last century the typical three types of movable bridges, bascule; swing; and vertical lift. Regardless of this, innovative solutions are increasingly taking form.

As they are of a complex combination of structural, mechanical, electrical and hydraulic systems, movable bridges have some issues and associated special requirements and are more prove to experience significantly higher deterioration and declining than regular fixed bridges. Thus, a good strategy design and maintenance requirements are of extremely importance.
In this sense, the elaboration of a specific and good technical approach of the design is needed. This approach aims at the conception (early stages) of the design, enhancing communication between the wide range of project members of every domains.

The Design Approach developed during this study establishes and revisits design requirements for movable bridges to minimize any potential problem that is susceptible to cause failure of the operation of a movable bridge. It helps, therefore, establishing the basic concepts to provide all bridge components (of different domains) with reasonable capacity against loads and operation reability.

1.2. OBJECTIVES

The aim of this work was to develop an enhanced approach about the conceptual and preliminary design of movable bridges. Different types of movable bridges and their design features were investigated in order to then enable the right approach to the design in question. It was then carried out a real case study (Great Yarmouth Third Crossing) as part of an internship done in the company Mouchel Consulting/WSP in Manchester, with the main purpose to provide a final recommendation for the three main types of movable bridges herein studied.

The main objectives of the present documents are:

- Make a brief historical context of movable bridges, identifying structural typologies and different main characteristics;
- Present the fundamental features and approaches of designing a movable bridge;
- Improve the ability of interconnecting the different fields considered in design;
- Describe in detail the case study of conceptual and preliminary design stages of a movable bridge over a navigable river.

1.3. APPROACH

Taking into account the objectives described before, the present work was organised in five chapters. The content of each of them is summarized below:

- Chapter 1 – Presents a brief introduction about movable bridges and their design, giving particularly attention to the need for a detailed knowledge of the unique specifications of these types of structures design. Identifies the main purposes of the dissertation and presents the chapters approach.

- Chapter 2 – Begins with a brief historical research of the construction of movable bridges relating with their construction time to the structural type and materials used. Afterwards, it is presented a learning of the different types of movable bridges and their most important features.

- Chapter 3 – Presents a methodology and a description of the most important details, issues and common mechanisms that have to be taken into account when a conceptual and preliminary design is carried out.

- Chapter 4 – From the information studied and developed in the previous chapters, a real case study is assessed regarding a conceptual and preliminary design, in the United Kingdom. In this chapter are described the difficulties of implementing the project design and the optimal approach that led to the final solution.

- Chapter 5 – Final conclusions and comments are made about the work developed here and some guidelines are also proposed for the development of possible future work.

In addition, this dissertation presents, an appendix with the preliminary calculations carried out and the results of a finite element model created on the program Midas Civil to confirm the feasibility of the final structure recommendation.

An appendix is also presented with some real examples of the different types of movable bridges described in chapter 2.

2

HISTORY AND TYPES OF MOVABLE BRIDGES

2.1. BACKGROUND

2.1.1. EARLY TIMES

It is thought that the first movable bridges to be built were in Ancient Egypt, circa 1855 BC, in the 12th Egyptian Dynasty. According to Edward H. Knight (1876), the first allusion to movable bridges was made in Egyptian monuments, such as palaces and temples, where there would be illustrative drawings of these bridges around castles and fortified cities. (Hovey, 1926) During the reign of Ramsess II in 1355 BC, the use of so-called floating bridges on the Nile River was already mention. (Mahmoud, 2003)

Around 460 AD, Nitocris, the queen of Babylon had a bridge built on the Euphrates River, one of the main elements of the Tigris-Euphrates system, which defined Mesopotamia. According to the stories of Herodotus, this bridge was built with pillars of stone blocks connected with iron and lead and spans of wooden platforms, wich would be removed at nigth to prevent the passage of people from both shores. (Hovey, 1926)

The tradition of building movable bridges was probably exported from these regions to Syria, with examples circa 1100, and then to Europe, but some examples are dated before the year 1000 in China. [Mahmoud, 2003] Otis Ellis Hovey, noted that most likely around the third century and into the middle of the sixth century, the Chinese have used movable bridges in their channels, as they have developed an unusual skill in engineering since very early. (Hovey, 1926)

Some centuries later, in 621 AD, in the Roman Empire, was built the first recorded movable Roman Bridge, by Ancus Martius. This was made up of planks of wood and according to some writers had a drawbridge. (Hovey, 1926)

The first and perhaps the most common movable bridges were, as already mentioned, pontoon bridges. These were mostly used in military expeditions and were built by piles of wood, with small vessels, tied together strategically so that they could be moved or swung to allow a navigable passage. An example of this is the Darius boat bridge over the Bosphorus Thracian River in Turkey, which connects Europe to Asia, and the Xerxes Bridge over the Hellespont passage, now called Dardanelles, in Turkey. (Hovey, 1926)

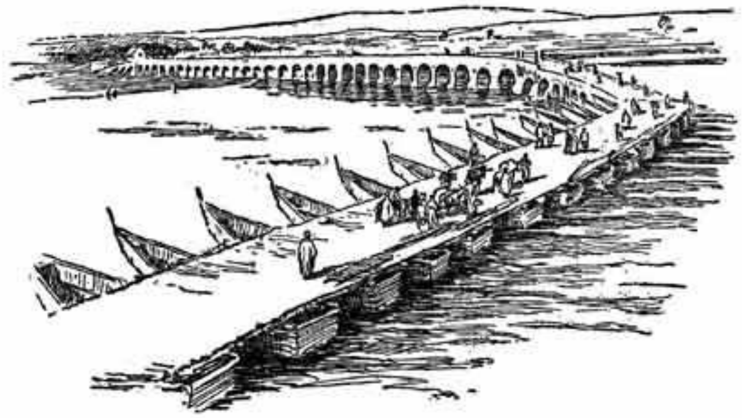
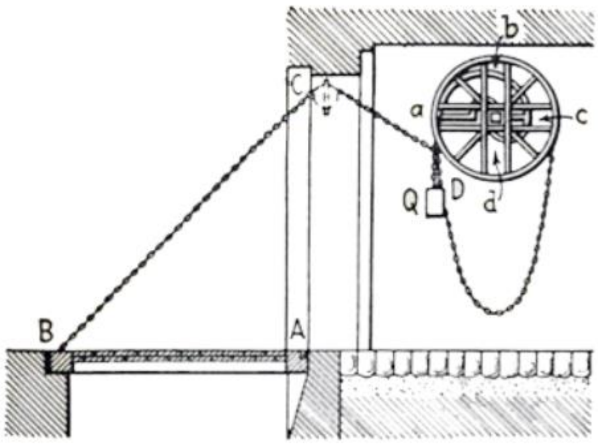


Fig. 2.1 – Sketch of Xerxes Bridge (Unknown, n.d.)

The Romans quickly developed the construction and design of fixed arch bridges, but it is difficult to trace the progress of movable bridges to the beginning of the Christian Era. (Hovey, 1926)

Later, during the Middle Ages, the drawbridge, described at the beginning of the chapter 1, was perhaps the most common bridge, being used as a passage in the closed position, but also when in open position, as a protective castle barrier. (Hovey, 1926) The most typical set up would be a lift movable bridge just outside the gate, comprising a wooden plataform with a pinered articulation at the edge of the gate. This would enable the plataform to rotate about this axle, making this type of bridge the processor of bascule bridges. This lifting, as can be seen in figure 2.2, was carried out throught ropes or chains linked to a reel, in a dramber over the entrance. Counterweights were sometimes used to balance the bridge and reduce the required force for this operation. (Koglin, 2003) A fairly ingenious alternative system can also be seen on the right-hand side to replace the previous conventional drawbridge. This mecanism works by a top frame (L) being raised, pivoted by a support (M) and connected by chains to the end of the bridge. The particularity of this bridge is the existence of a counterweight falling in a curved trajectory to a lower level. The structure would always be in balance throughout its opening process. This will later be known as the Bélidor type bridge. According to Ottis Hovey, the oldest bridge of this style is the bridge in the fortress of Bonifacio in Corsica.



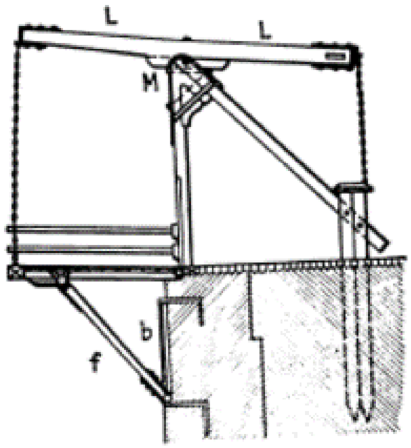


Fig. 2.2 – Typical ‘draw’ Bridge (Berger, Healy, & Tilley, 2015)   Figure 2.3 – Oldest ‘draw’ Bridge (Hovey, 1926)

Since the 14th century until the end of the 16th century, in the Renaissance, which was marked by very distinguished transformations in the area of culture, science, art, economics, politics and religion, was born a period of great human minds and discoveries. Movable bridges were also an expression of this era and did not go unnoticed, where it is noteworthy the existence of several sketches on this subject, authored by one of the most remarkable minds of that time, Leonardo da Vinci. Many of these sketches are still preserved, and were published in the '*Codice Atlantico*'[1]. Figure 2.4 show sketches of Da Vinci himself and it can be observed that, in the year 1500, the engineering of movable bridges, more properly swing bridges, was already well advanced. Figure 2.4 shows a design of swing bridges in which the rotation was operated by hand winches, through ropes connected to snacth-blocks. (Hovey, 1926)

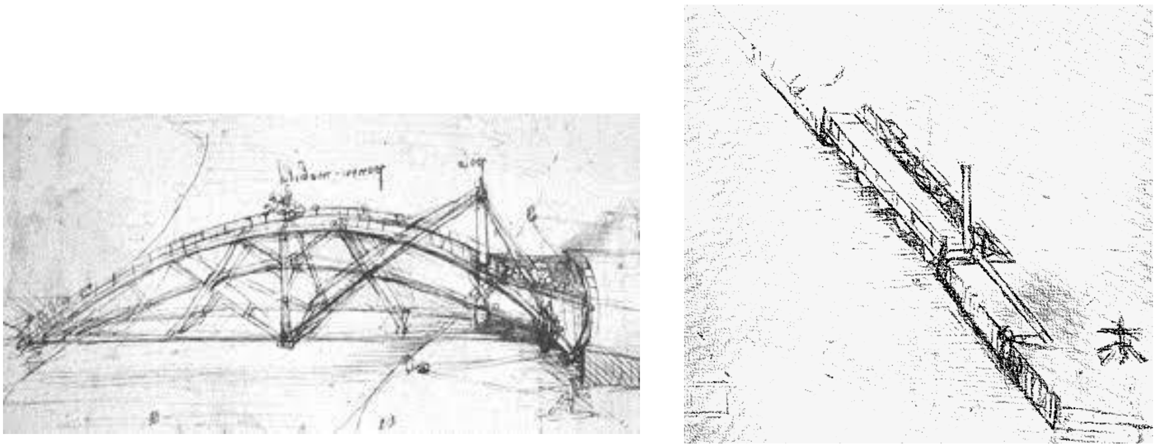


Fig. 2.4 – Da Vinci sketch of Swing Bridges (Hovey, 1926)

Some sketches also show the development of some ideas of vertical lift bridges with counterweights. This period was very rich in terms of bridge construction developments and many of the projects and constructions undertaken at this time served as basis for many of the modern bridge constructions recently used in Europe and America.

Further types of movable bridges designs were developed and great skill was demonstrated by various engineers throughout their (designs) gradual evolution to modern times.

A new era of engineering was born at the invention of the steam engine by James Watt in 1769 and the introduction of the steam locomotive by George Stephenson, in 1829. These two inventions marked the production of energy and were responsible for the fast evolution of almost all forms of engineering. Movable bridges were no exception and their design and construction had a tremendous progress. Many types of movable bridges were developed at the time, but the swing type was the one that grew more in popularity and, hence, in development. Since creating power was expensive, it was thought that the swing type bridge was cheaper. This, because these bridges were of a simple design and were balanced to both sides, from an axis of rotation, located under its center of gravity, not being necessary the operation of raising and lowering. (Koglin, 2003)

[1] Codice Atlantico is the biggest collection of drawings written by Leonardo Da Vinci and is found in the Ambrosiana Library of Milan

Although the inventions referred previously, resulted in a vast increase of movable bridges construction, the followed boom of the railroad industry, in the mid-nineteenth century and the lorry industry after the First World War danwed a new start. (Ryall, Parke, & Harding, 2000) The fast pace of industrial growth has made it essential to find a way to cross rivers and active channels used by a variety of vessels of different sizes, so that road and rail traffic could travel without permanently interfering with the flow of any kind of traffic. This was a particular problem in expanding cities with important navigable waterways, so movable bridges were the answer. (Sloan, 2004) This industry boom has provided a boost to the metal and the development of the mass-manufacturing processes. This allowed the construction of lighter and stronger spans, longer bearings and more powerful engines. (Hall, Unknown)

2.1.2. Modern Times

At the beginning of the 17th century, knowledge of the rotation of swing bridges through pivots with centre bearings supporting devices was already visible. But it was only at the beginning of the 19th century that british engineers developed pivots with rim bearing supporting devices which were capable of withstand the immense weight of large swing spans. Due to its simplicity, reliability and economy, the type of centre bearing prevailed over the more complex design and by the third decade of the 20th century, this type almost replaced the rim bearing . At that time, engineers came to the conclusion to appreciate the advantages of this type of bridges over the most diverse forms of movable bridges. (Unknown, Unknown)

As Otis Hovey said in 1926, "when there are no restricting circumstances, a swing bridge is the simplest, best, and most economical type in first cost and maintenance". Not all engineers agreed with Hovey's statement of superiority of swing bridges. The bascule bridge had many followers throughout history. In the 20th century, George Hool, professor of Structural Engineering at the Wisconsin University, vigorously supported the benefits of this type of bridge in both its 1924 and 1943 editions of Movable Bridges and Long-Span Bridges. This preference was mainly due to the fast opening of the bridge, causing the ship to pass through the river/channel more quickly. (Hool & Kinne, 1943) Despite supporting the swing bridge, Hovey acknowledged that the bascule bridge was superior when many parallel bridges had to be upright and when the waterways were too narrow. (Hovey, 1926)

The oldest construction of a modern bascule bridge dates back to 1894 with the construction of the Tower Bridge in London and Van Buren in Chicago. A number of bascules bridge designs have been developed and patented over the following decades. According to bridge engineer J.A.L Waddell: "they [the designs] are scientific, and they represent, probably, the best and most profound thought that has ever been devoted to bridge engineering", as he also patented a type of bascule bridge. (Waddell, 1916) During this period, two types of bascules bridges prevailed, the trunnion bascule and the rolling lift bascule. The trunnion, in its simplest forms, evolved from medieval drawbridges and was developed by European military engineers in the early 18th century. JAL Waddell states in his work “Bridge Engineering” from 1926 that the bridge Michigan Avenue, in Buffalo, New York was the first major bascule bridge to be constructed. This type later evolved in the 19th and 20th centuries to two variations, the simple trunnion or "Chicago" and the multiple trunnion or Strauss. (Hovey, 1926)

The simple trunnion, patented by the Chicago Bascule Bridge Company, was basically an improvement of the counterweights mechanism. The design of multiple trunnions was far much more complex, which in addition to the main, contained three secondary trunnions, and all connected by supports that form a rectangle when the span is closed and a parallelogram when the span is opened. (Hovey, 1926)

The rolling lift, maintaining the natural movement of the upward-swinging motion, evolved by adding an additional movement - the span retreated from the opening as it was lifted, thus providing even more clearance for navigation. This has been achieved by attaching the span to a beam segment, which tilts the span upward as it retreats in its track, simultaneously. Two early 19th century French bridges, built in Havre and Bregere, were the predecessors of this type of bridges.

At the end of the 19th century, two variants of the rolling lift type were patented, Scherzer and Rall, which will be studied in detail later in section 2.2. Developed in 1893 by William Scherzer, American engineer, held twelve patents for various variations of this type of bridge since 1893 to 1921, which became the most popular of all types until 1916. The first exemplar of this bridge was the Van Buren Street Bridge, located in Chicago, followed by many others, as this design resulted in the replacement of many movable bridges in England docks, such as Liverpool, Birkenhead and London. This movement occurred since its structure allowed to cross much larger channels than the other existing types at that time.

The Rall system, created and patented by Theodor Rall in 1901, was the other variant to be designed. The company who held the construction rights was Strobel steel Construction. One of the few bridges still existing of this form and perhaps most well-known is the Broadway bridge, in Portland, United States of America. This is also the largest span ever built of this type of system.

Until 1908, little progress was made in the building of vertical lifting bridges. As of this date, and approximately in the next two decades, was a great deal of interest from bridge engineers in this type of movable bridges, who have built about 70 movable vertical lifting bridges in America alone. This interest was held to be possible because of the varying amount of advantages these bridges entail. (Hool & Kinne, 1943)

2.1.3. Last Tendencies

In the last two decades, the movable bridges technology has undergone changes very quickly, due mainly to the introduction of hydraulic machinerys and automatic controllers. Through these developments, it was possible to operate bridges easier and safer, using smaller machinery. (Mahmoud, 2003) But despite the great evolution of the constructive technical part, the aesthetics of modern bridges falls short. During the latest half of this century, no substantial innovation was made in this aspect, with several existing designs being characterized by repetitive and unattractive solutions. (Mahmoud, 2003)

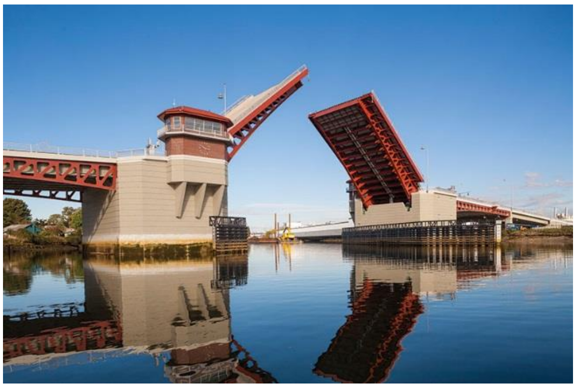


Fig. 2.5 - South Park Bridge (Lane, 2016)

Although this unattractive tendency, in recent years, some projects have distinguished themselves by their exceptionality, giving new hope to the design of movable bridges. This is mostly due to the use of advanced software available and sophisticated modeling, such as finite element models. The possibilities of determining the natural frequencies of the structures, with the availability of new high-performance materials, are allowing a world of new innovative solutions with incredible structural schemes, for instance the use of highly slender spans. (Mahmoud, 2003)

More recently, it has been proven that kinematics plays a much more important role in the design and analysis of this type of bridge than previously thought. Nowadays, there are already simulation models for complex kinematic mechanisms, allowing the engineer a much better control of the structural system with the type of elevation scheme. (Mahmoud, 2003)

Fig. 2.6 - Gateshead Millennium Bridge (Unknown, www.resimhayattir.com, n.d.)

2.2. TYPES AND FEATURES OF MOVABLE BRIDGES

As previously mentioned, movable bridges have had a huge record and evolution through the history of bridge engineering.

According to a study by C. C. Schneider (former president of the Society of American Engineers), in 1907, to expose movable bridges and establish their specifications, movable bridges were classified into six main types:

- Bascule Bridges
- Vertical lift Bridges
- Swing Bridges
- Transporter Bridges
- Retractile Bridges
- Pontoon Bridges

It should be noted that some of these bridges can be subdivided due to their unique characteristics and are not restricted to those presented above, since they can be created to meet the specific conditions of the place in question.

In this paper work it only will be studied the three main and common types of movable bridges: Bascule, Vertical lift and Swing bridges.

2.2.1. Bascule Bridges

The bascule bridges are based on the simple principle that if one end of the span is raised, the other one has to be lowered. The term bascule is generally applied to any type that moves through a fixed or movable axis, and those that move through a circular segment of beam.This occurs, by pivoting on a horizontal axis, at a certain angle. This pivot should be close to its center of gravity so that the weight on one side can be balanced by the weight of the other side. (Koglin, 2003)

The deck of these type of bridges may consist of one or two leaves that is a single span, or two symmetrical ones, which when in the close position, engage one another, ensuring that the two work together as one, causing their final deflection to be the same, when loaded. (Hovey, 1926)

The end of the bascule span is called the toe of the leaf, and the part of the span near the pivot point is called the heel of the leaf. This point, adjacent to both the approach span and deck, is supported by the bascule pier. Normally the counterweight is at the back end of the leaf and serves to balance the leaf about the trunnion. This is placed outside of the pier so that it is exposed. Such a design is used because is advantageous in that it minimizes the width of the pier and can resist uplift when there is traffic at the span. The trunnion, previously mentioned is used on many of these bridges and basically is a pivot on a shaft. First associated with cannon, an important military development, is normally used to describe a cylindrical protrusion used as a pivoting point that rotates while supporting a load. This is frequently thought incorrectly because the trunnion only rotates a fraction of a turn and never the full rotation. (Koglin, 2003) The operating machinery must be capable and ample enough to overcome the friction of all the trunnions and joints and support the mass of the moving leaf, counterweights and the rest of the structural members. Normally the weight of the counterweights are from two to three times bigger than the leaf, therefore be required a proportionately power by the machinery.

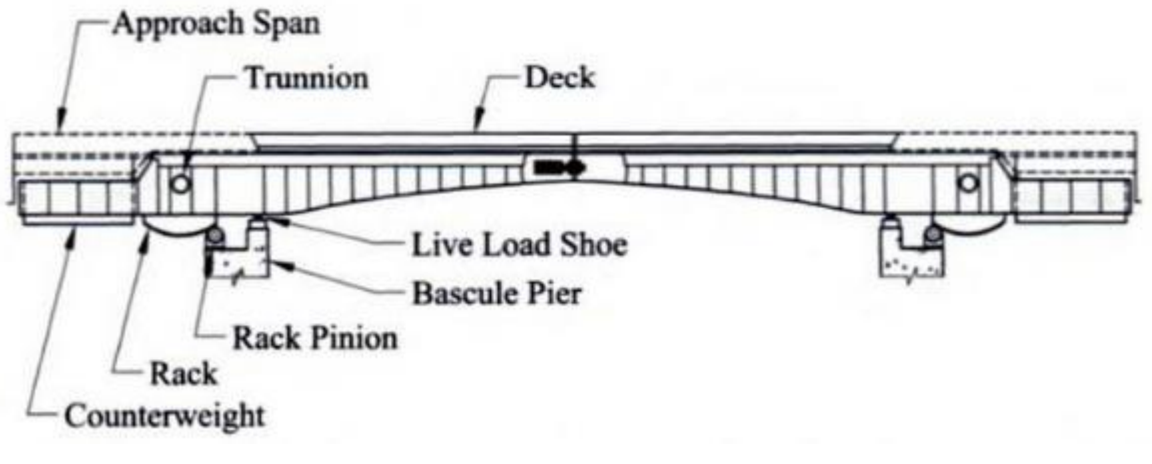


Fig. 2.7 - Typical Bascule Bridge – Trunnion type (Koglin, 2003)

The bascule bridge had a large development because of its many advantages, mentioned next:

Operation - Bascules do not need to be fully opened to allow small boats to pass, so the time for the operation is proportional to the sort of opening. This, comparing to swing bridges, that require a complete 90 degrees opening to let vessels pass, regardless of vertical clearance, can be a huge factor of decision depending on the traffic conditions.

It is also important to state that the bascule does not block the channel during operation, therefore it does not require occupation of river frontage like swing bridges. In narrow waterways, this is an essential issue. One of the most important vantages of bascule bridges is the no limitation of air draught when in open position.

Pier Considerations - For certain locations with narrows channels the piers can be the main consideration as these are smaller that the ones for swing bridges. The fendering system needed for protection of the piers and auxiliary spans can be shorter than for swing bridges. Bascules with underneath counterweights need a counterweight chamber (called main pier) below deck level.

Costs - Bascule bridges are very economical for medium range spans (10-50m) but can be built for any length.

Superstructure - The superstructure area limited to navigation span only and normally has a minimal visual impact when in closed position and significant visual impact when raised. Mechanically simple in single leaf form with no locking system required. As the leaf is raised more installed power is required than other movable bridges types to overcome the wind loading.

2.2.1.1. Types of Bascule Bridges

Bascules bridges can be categorised as in three main classes - rolling type, trunnion type (Chicago) and Strauss type. Any of these types may have either a single or a double leaf, being that for railroad traffic the single leaf is preferable, for it can be made to act as a simple span when closed and so a greater rigidity is guaranted.

1) **Rolling bascule bridge**

Scherzer Bascule Bridge

Rolling bascule bridges are generally refered as 'Scherzer Bascule bridges' due to is inventor and patents, and it is characterised by having at one end a cylindrical curved part that rolls upon tracks (usually in the form of a heavy girder) when the bascule leaf rotates open or closed. These curved part are usually designed by 'segmental girders'. Slippage between the curved part treads and the running tracks is prevented by a teeth meshing that engage one another. (Berger, Healy, & Tilley, 2015) Due to their large size, many Scherzer rolling bridges incorporate counterweights made of cast iron or another dense material to reduce the size and the cost of the structure. This allows a reduction of wind resistence because it is possible the use of a smaller diameter of treads and consequently smaller segmental girders. (Koglin, 2003)

This type has the distinct advantage over other forms of movable bridges in making the navigation channel free for boats more quickly. Thus, since it translates away from the channel as it rotates open, the angle necessary to provide the same clearance as other bridges is much smaller. (Koglin, 2003)

Three common types of Scherzer bascules include the deck double-leaf, the half-through single –leaf and the through single-leaf:

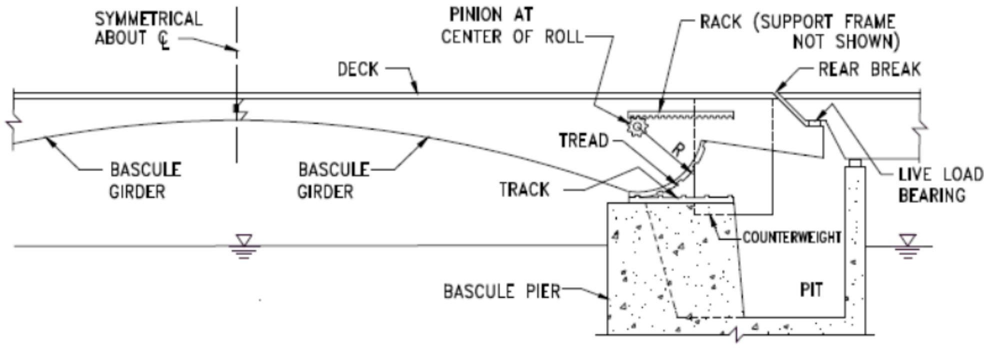


Fig. 2.8 – Double-leaf Scherzer bascule Type (Berger, Healy, & Tilley, 2015)

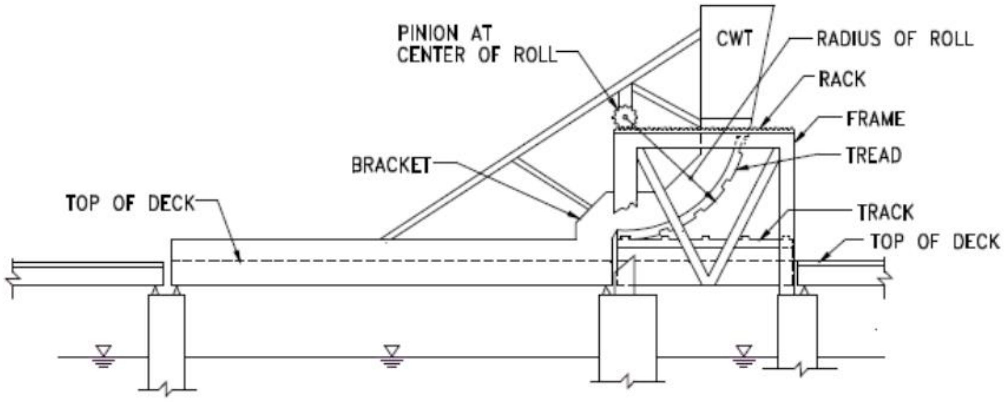


Fig. 2.9 – Half-through single leaf Scherzer bascule Type (Berger, Healy, & Tilley, 2015)

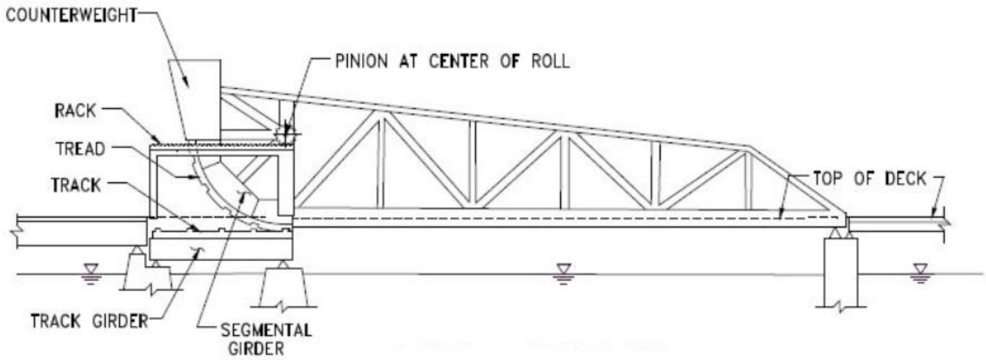


Fig. 2.10 – Through truss single leaf Scherzer Bascule Type (Berger, Healy, & Tilley, 2015)

Rall Bascule Bridge

Another variation of the rolling lift type, developed by Theodore Rall, is the Rall bascule bridge that combines rolling with longitudinal motion. The key feature is a trunnion that is set inside a roller that moves along a track.The combined movement can be explained by the example refered below, which is a diagram of an electrical railway bridge over the Illinous River, ate Peoria, Illinous.

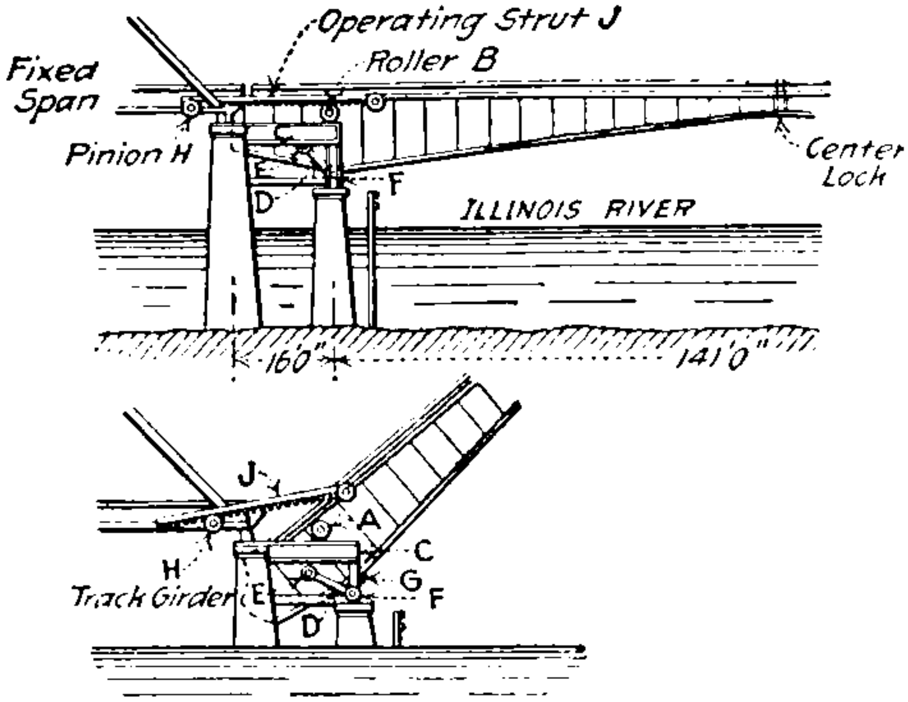


Fig. 2.11 - Rall bascule bridge, Illinois River (Hovey, 1926)

The counterweight of the main girder which is connected to the pier by the swing strut D and turns on pins, does not required a pit to received is tail, since the centre of rotation is to far from the pier.

One of the many features of the Rall type is that, when closed, is possible to remove and replace the pivoted rollers A, as they are released of all loads. The retreating motion of the leaf allows a minimum span length to achieve a clear waterway, however the shifting of the centre of gravity by the rollers disturbs the foundation pressures. Also, it has to be taken in account the weight contact between the rollers and the tracks and the friction between them, so it is necessary the finest design and material of this elements. (Hovey, 1926)

2) Simple Trunnion or 'Chicago' Bascule Bridge

Simple trunnion bascules main feature is the use of a system of heavy counterweights mounted on a frame at the end of the span. This allows the redution of the size of the mechanical power system components required to operate the bridge and in case of failure a marge of safety.

One of the characteristics of this bridge is that the part of the leaf that extends over the water is much longer than the part that is mounted on a frame. During the opening of the span, the counterweights and leaf weight are supported by trunnions carried in trunnion bearings attached to the piers and located approximately at the center of gravity of the entire mass. (Hool & Kinne, 1943)

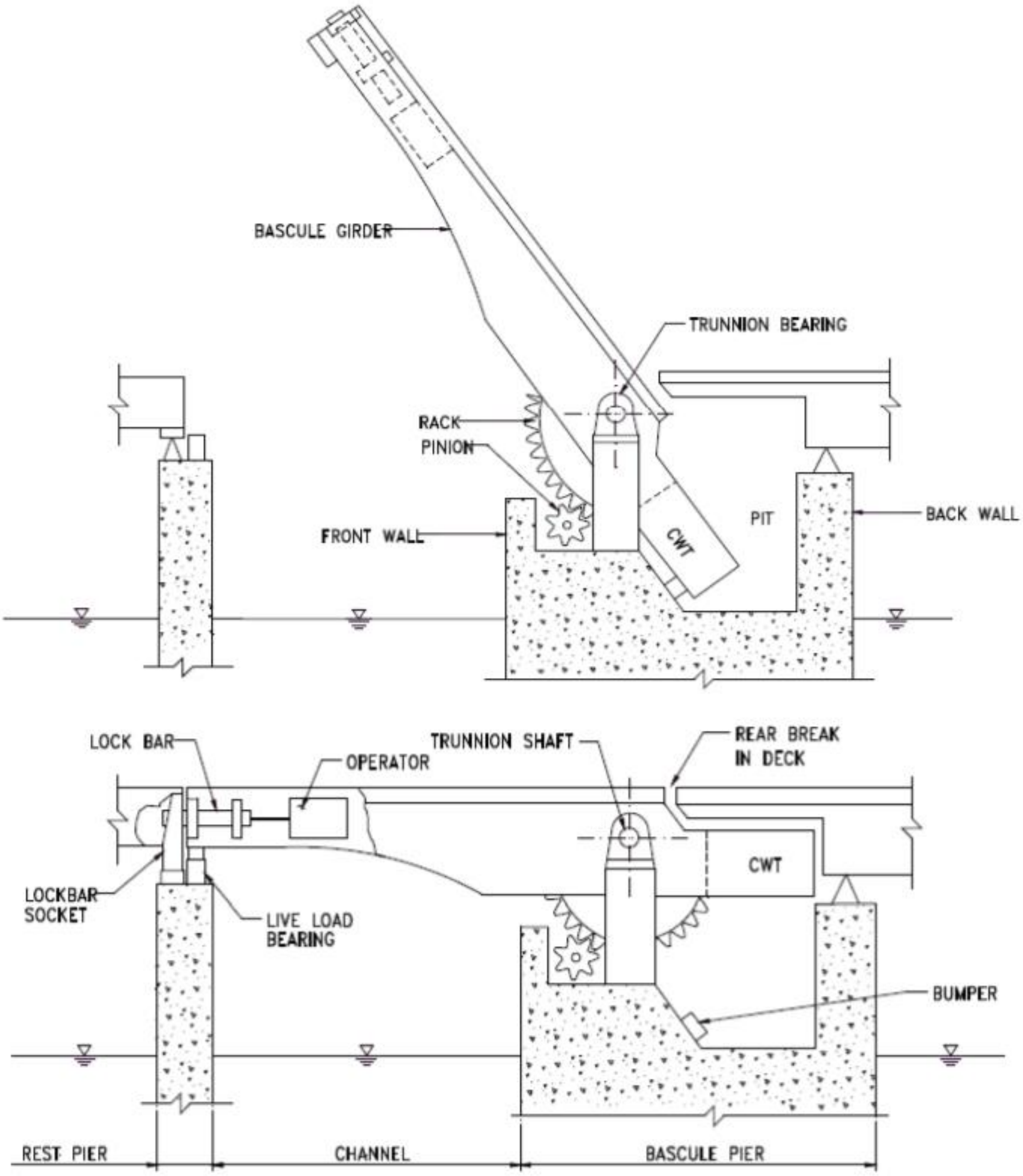


Fig. 2.12 - Simple trunnion bascule bridge (Berger, Healy, & Tilley, 2015)

When the bridge is being operated, power is transmited to pinions that rolls on curved racks in one direction to open and on the other to close it. The trunnions bearings can be attached directly or indirectly to the masonry of the piers, being therefore possible the inexistence of piers. The frame were the counterweight fits should have elastic bumpers to absorb the shock, as showed in figure 2.12.

A big disadvantage of this type is that when it is needed a medium or long span bridge close to water level, is required large bascule pires with deep pits.This type has been topic of various discussions and research for the reason that it is possible to have various different systems capable of supporting the trunnions bearings. Although these counterweights are expensive to construct and requires a large area of space, the Simple trunnion bascule is strong and simple in operation, hence is one of the most used types of all. The most recognised development of this type was the evolution of the elevation system to a hydraulic trunnion.

3) Strauss Bascule Bridge (Multiple Trunnion)

The concept of the bridge that gave right to the multiple trunnion design was the idea of a remote counterweight system that connects indirectly to the tail end of the span. There have been more bascule bridges built from the Strauss designs than any other single type of bascule. The Strauss Bascule can be categorised:

Heel-Trunnion

Figure 2.13 ilustrates the overwall operation of the Strauss hell-trunnion. This has a distinctive feature of having an overhead gyratory counterweight. The points D-E-B1-B2 form is a parallelogram.

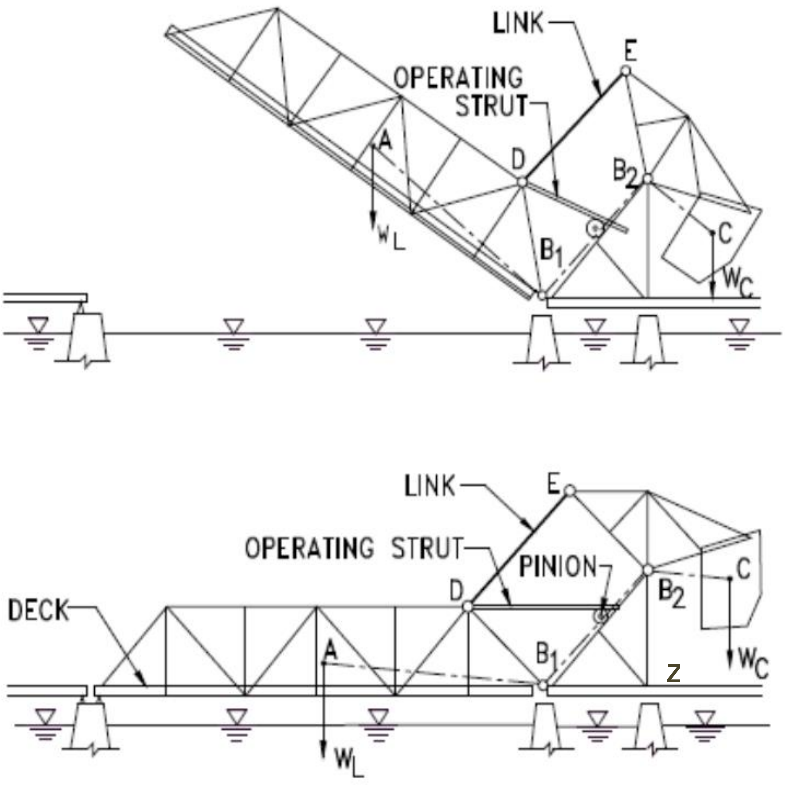


Fig. 2.13 – Strauss Heel-Trunnion (Berger, Healy, & Tilley, 2015)

The moving of the leaf is operated by means of a strut that is articulated to the trusses at D and extendes with a rack engaging to a main pinion. When the bridge is in the closed position the struts, being heavy, upholds the span, but when it starts to rise, this act as cantilevers, and tend to assist the counterweight and sustain the open position. In this movement the trunnions that form a parallelogram folds up and the upper arm lowers, causing the counterweight to lower as well. This points that form the parallelogram form, D-E-B1-B2, are under heavy stress during this process.

As it will be demonstrate ahead, while the underneath and overhead counterweight have only one pier supporting the weight of the span and the counterweights, this type has 2 piers.

Overhead Counterweight

Figure 2.14 represents the Overhead Counterweight Strauss type. This type has the distinguishing feature of having the counterweight placed above the road level, being of advantage for locations that have the water level close to the road level. These are also used when the appearance of the bridge is not the first considerating, and when the pier cost has to be minimised. Same as the Heel Trunnion, some of the trunnions in this bridge form a parallelogram, which in this case are trunnions B-C-D-E. One particular feature of this is that the main trunnion B can be placed at any point desired, if this relation is maintained.

The main disadvantage is the quantity of intermoving parts needed and the hinged and swinging counterweight.

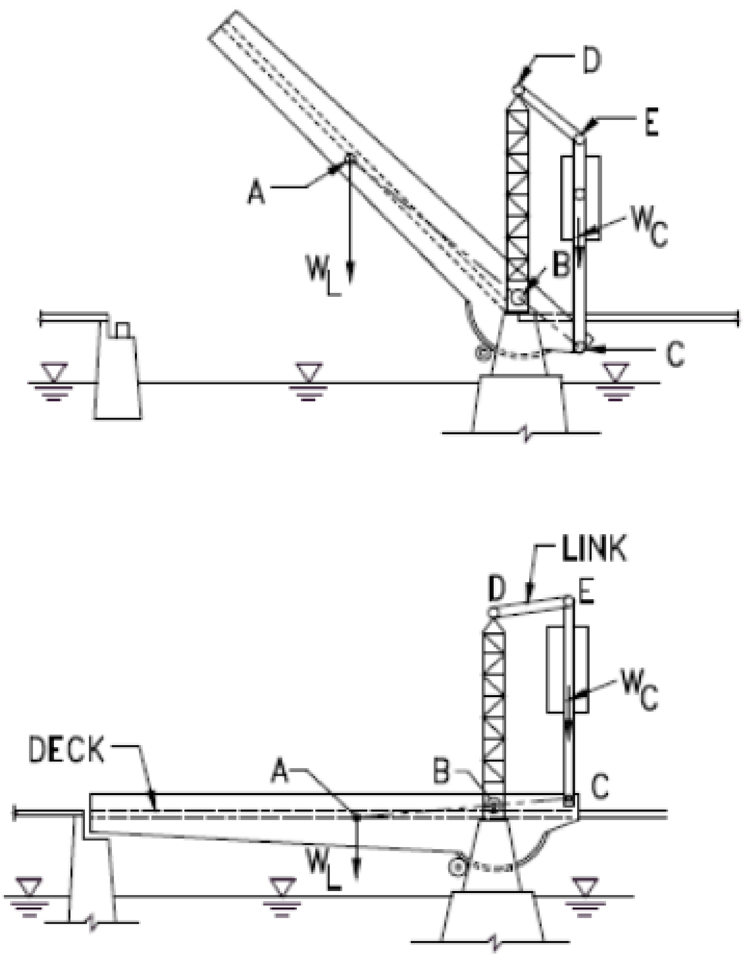


Fig. 2.14 – Strauss Overhead Counterweight (Berger, Healy, & Tilley, 2015)

Underneath Counterweight

The Underneath Counterweight Strauss type operation is showned in the figure 2.15. The principle of this type is the same as the described above for the Overhead Counterweight, but with the counterweight and link located underneath the road level. This is normally used when exists ample clearance between high water level and road level.

One feature that has been responsible for many discussions throughout the years is the existence of the link connecting the counterweight with the trunnion tower. Some says that for a small angle of opening the friction in the counterweight trunnion bearings may not permit the rotation of the support in order to C-E remains vertical. During the opening of the bridge the angle increases and the moment applied to C would increase as well, being that if exceedes the bearing frition moment the counterweight would swing without any restriction, which could cause the inability to control the moving leaf. (Berger, Healy, & Tilley, 2015)

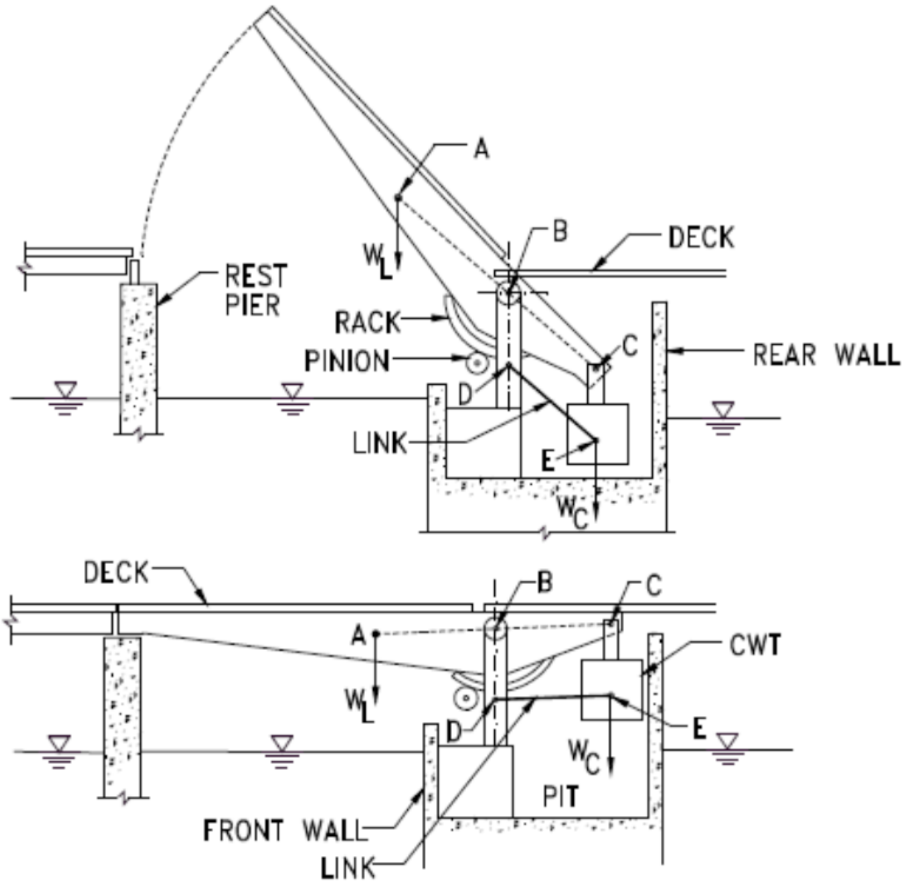


Fig. 2.15 – Strauss underneath Counterweight (Berger, Healy, & Tilley, 2015)

4) Other Types of Bascule Bridges

During the many years, other types of bascule bridges were developed. Some of them were successfully introduced but some others were easily put aside in favor of the most common type’s descridbed above:

- Belidor
- Balance beam
- Roller Bearing
- Brown
- Page
- Semi-lift
- Dutch

It is demonstrated in Table 2.1 the main features of the usual types described before.

Table 2.1 – Summary of features of bascule bridge types

Type of bascule bridge		Advantages	Issues
Rolling	**Scherzer**	- Provides large angle of openings - Allows for shorter span lengths - Allows for smaller counterweights, resulting in smaller piers	- Tends to move transversely during operation - Requires special detailing of the track - Initial alignment challenges - Requires additional machinery (i.e. tail locks) - Operating machinery mounted to the movable span
	Rall	- Provides large angle of openings - Allows for shorter span lengths - Rollers can be removed for repairs when the bridge is closed (relieved of all load)	- Higher level of friction between the elements - Weight taken by line contacts between the rollers and their tracks
Simple Trunnion or 'Chicago'		- No channel obstruction - No visual impact - Trunnion on fixed axis - Provides reliable operation - Span opens and closes same each time	- Requires longer span, deeper pier compared to rolling lift or other counterweight types - Requires special counterweight detailing to maximize angle of opening and avoid interference with trunnion - Transverse horizontal axis trunnion supports

Strauss or Multiple trunnion	**Heel-Trunnion**	- Small substructure - Allows for low profiles without counterweight pit - Pivot/Trunnion closer to navigation channel, providing same channel as simple trunnion with shorter leaf	- Stress reversals at rocking truss and counterweight link - Overstressed pins and rocking truss
	Overhead Counterweight	- Small substructure - Allows for low profiles without pit	- Inadequate counterweight tower on some versions
	Underneath Counterweight	- Good architectural appearance	- Excessive friction in the counterweight linkage and trunnion bearings induces repetitive bending moment in the counterweight hangers – especially at small angle of openings - Adequate height to allow the counterweights to swing

2.2.2. Swing Bridges

Swing bridges are those who can provide a navigation channel by rotating about a vertical axis by a horizontal plane, normaly 90 degrees. This movement is possible by pivoting on a central pier through bearings connecting the deck and this pier, making it a pivot point. The movable span of a swing bridge - also called draw – is designated of bobtailed or unequal-armed when the arms are not of equal length and designated of symmetrical or equal-armed when the arms are of equal length (with the pivot point in the midle of the draw).

These, comparatively with other type of movable bridges, like bascule and vertical lift, are not lifted, so the lift mechanism is not equilibrated by gravity, being required a device to stop the span at the right position, i.e in the direction of the channel traffic. (Koglin, 2003)

Operation - When in open position ther is no limit on air draught and the visual impact is considered minimum. The wind load is not as severe as other types of bridges, so is require less installed power and therefore more high efficiency.

Superstructure - Depending on the length of the crossing channel, swing bridges can be built with one or two arms so they can be used for all span lengths - 10 until 300m. The tail or backspan are typically 30 – 40% of the main span, so it is needed a longer superstructure comparing with other types of movable bridges.

The principle characteristic noticed on these bridges is the necessity of having a big area to store the moving span when in open position. In the case of a two arms swing bridge, this has be sited in the midle of the navigation channel, thus reducing the length for navigation and turning the maintenance task more inaccessible and difficult. As normally, the superstructure and substructure are kept above the river water level, so a collision protection is needed along the full length of the superstructure.

Although there is no need for counterweights in the design of most swing bridges, it is required a wedging system, which is mechanically more complicated than other types, and thus potentially more labour intensive in maintenance.

Swing bridges are categorised according to their type of bearing - centre bearing, rim bearing, combined-bearing, slewing-bearing and pontoon-supported swing bridges. The most common types and described here are the first two.

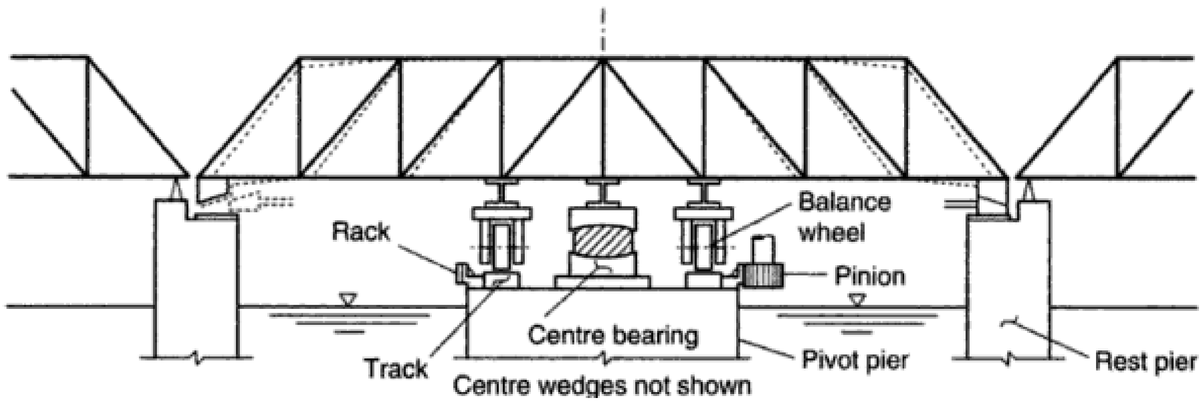


Fig. 2.16 - Centre bearing swing bridge (Ryall, Parke, & Harding, 2000)

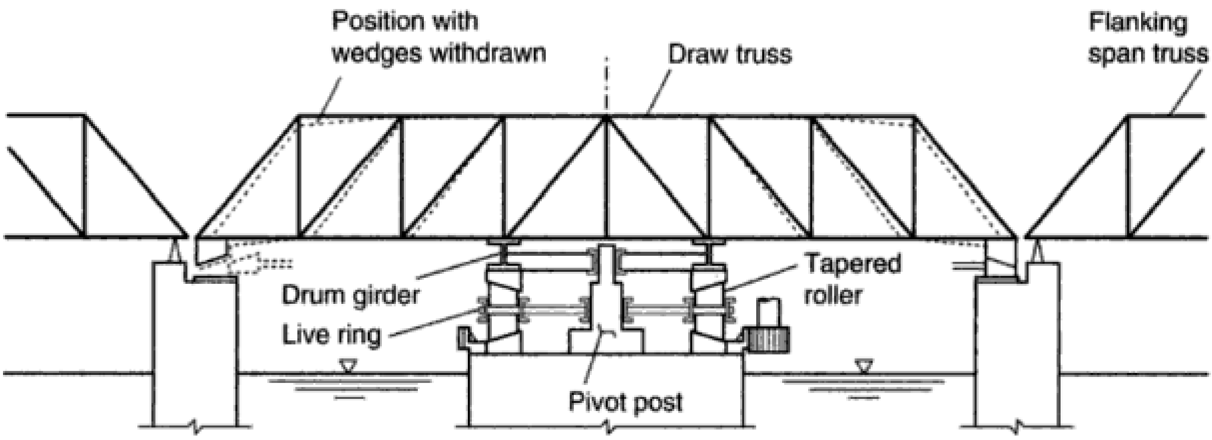


Fig. 2.17 - Rim bearing swing bridge (Ryall, Parke, & Harding, 2000)

On centre bearing swing bridges, when the bridge is in operation, all the dead load of the moving span is supported by the pivot bearing, i.e the span rotates on a single bearing support. The rotation of this type can be carried out by means of mechanical or hydraulic machinery.

(a) (b)

Fig. 2.18 – (a) Mechanical centre bearing (b) Mechanical rim bearing (Birnstiel, Bowden, & Foerster, 2015)

On rim bearing swing bridges, when the bridge is in operation, all the dead load of the moving span is supported by a serie of rollers on a circular track. The diameter of this circular track is usually around the same tranverse size as the outer swing span. This type is normaly used for long span or heavily loaded swing bridges.

2.2.3. Vertical Bridges

Vertical lift bridges are those capable of raising their deck vertically, keeping it horizontal (parallel to the water line), thanks to two or four side towers on each side of the bay in question. The lifting process is relatively simple, by means of a roller mechanism in the towers that go through cables, fixed to the span, which pass over the towers and are fixed to counterweights. These cables, through mechanical machines, can move upwards and consequently the counterweights downwards. The counterweights can be sited at the top of the towers externally or internally and ensure a balance of the system and minimize the amount of power require for the bridge lift process. This process is equivalent to a lift mechanism of a building.

This type of bridges has certain advantages over other bridges:

Simplicity - The usual bridge lift does not have much effort in its design and construction, in relation to other bridges. The complicated details are few and/or not complex.

Length of Span - These bridges can have longer spans and be more economically constructed than other types of movable bridges.

Lifting - The vertical lift bridge has the ability to be partially or totally raised, depending on the type of vessels, reducing the time of crossing of the navigable channel. The time required to the complete lift operation, i.e. opening of the bridge plus boat passage and lowering of the bridge is less than for the swing bridges. This occurs because there is no channel/river obstruction time in the operation.

Construction costs - Larger spans have less construction costs in this type of bridge.

Colision with vessels - In case of a collision with the bridge, at the time of elevation, it would be very likely to suffer damage only through the masts or chimneys of the boats and the pilot's tower. While, in the case of bascule and rotating bridges, the span would be smashed. Professionals say that the time for reconstruction would be shorter than for other bridges.

Interchangeable spans - In case of a bridge with several spans and in which the river is quite likely to change its water flow, this type of bridge can be constructed in such a way to be possible the movement of its towers, counterweights and machinery from one bay to another. This is possible because the spans are all constructed equaly.

Traffic - The weight of the counterweights must be equal to that of the deck, so heavier materials can be used in the deck, therefore heavier traffic can be used.

Despite the many advantages that this bridge entails, the biggest disadvantage in relation to other bridges is the restriction in height. As an elevation of the deck is made, it is suspended above the channel, which causes air draught limit.

Vertical lift bridges are normally labelled by the arrangement of the drive machinery:

Tower drive vertical lift

The tower lift system uses machinery located at each tower to raise the span. In this case, in addition to the normal lifting mechanism, it is often needed to implement differential height control equipment of the tray, as each equipment operates individually in each end, allowing the tray to be tilted. The equipment is located at the top or bottom of the towers, being the force required for the elevation transmitted by the cables to the counterweights usually by winches. (Berger, Healy, & Tilley, 2015)

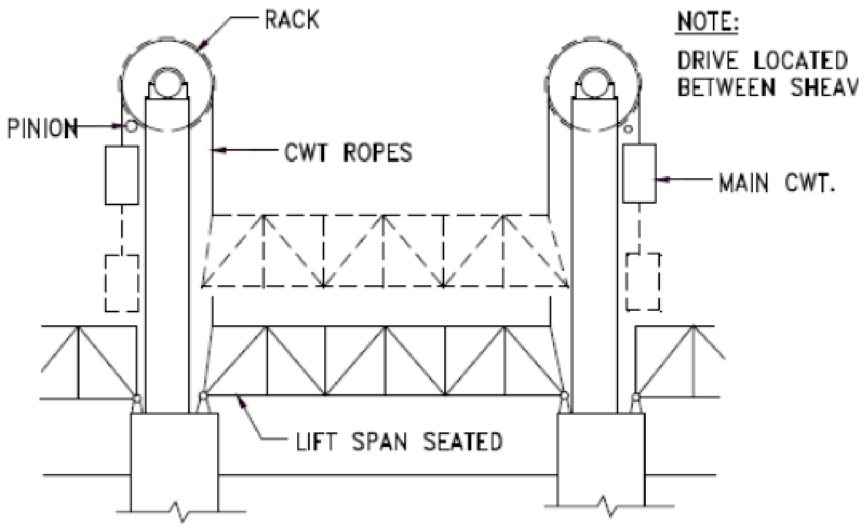


Fig. 2.19 - Tower drive vertical lift system (not to scale) (Berger, Healy, & Tilley, 2015)

Span drive vertical lift

Span drive vertical lift are categorised by having the drive machinery loacted on the span, normally at mid-span. The counterweights balance the weight of the span, transmitted by ropes, however for large span bridges this balance is difficult due to the difference in weights. Therefore, an auxiliary counterbalance system is needed to mitigate this difference. (Berger, Healy, & Tilley, 2015)

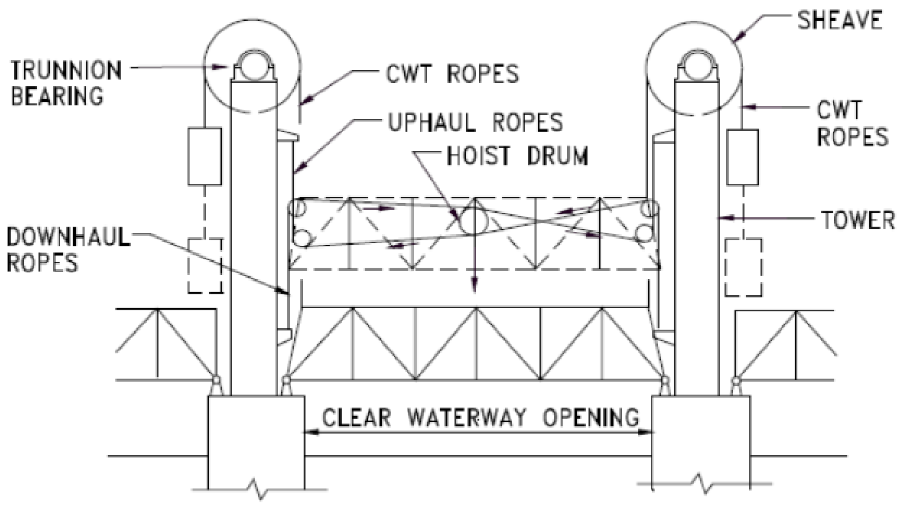


Fig. 2.20 - Span drive vertical lift system (not to scale) (Berger, Healy, & Tilley, 2015)

Connected tower drive vertical lift

These type of bridges are most suitable for small span bridges, so there is no need for an auxiliary counterweights system. The machinery is mounted at the top of the span, as can be seen in the figure below, but its positioning may vary depending on bridges.

The mechanism works by means of a force received by the drive, which makes the pinions to rotate engaging the racks attached to the sheaves, causing them to rotate as well. These then transmit by friction to the ropes of the counterweights, thereby raising and lowering the span.

This type of mechanism has the advantage that there is no inclination of the tray due to the position and connectivity of the different equipments. In spite of this, with time it is possible some misalignment or stretching of the ropes, causing inclinations to occur, being necessary for that reason a maintenance of the equipment. (Berger, Healy, & Tilley, 2015)

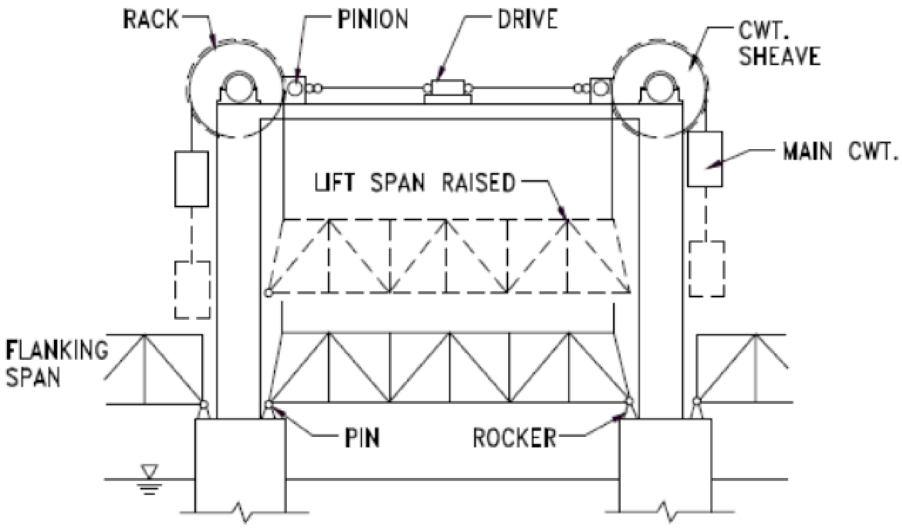


Fig. 2.21 - Connected tower drive vertical lift system (not to scale) (Berger, Healy, & Tilley, 2015)

Pit drive vertical lift

Characterized by the fact that the lifting mechanism is not visible when the bridge is in low position, it is used when there are aesthetic restrictions and for low lifting heights. The towers supports are not required and the ascent is driven by hydraulic cylinders installed in wells, located inside the pillars. These are normally below the water level of the channel / river. This mechanism is achieved through lifting posts (see figure 2.22), i.e. fixed legs inside the pillars that extend and collect. These posts, guide the movable span during the movement and resist the horizontal forces applied to the span when it is in opened position. The number of lifting posts depends on the width of the span.

This type of bridges are also able to withstand counterweights and sheaves to decrease the weight to be raised, however these features are uncommon because the use of mechanical cylinders with electric motors provides higher lifting power. (Berger, Healy, & Tilley, 2015)

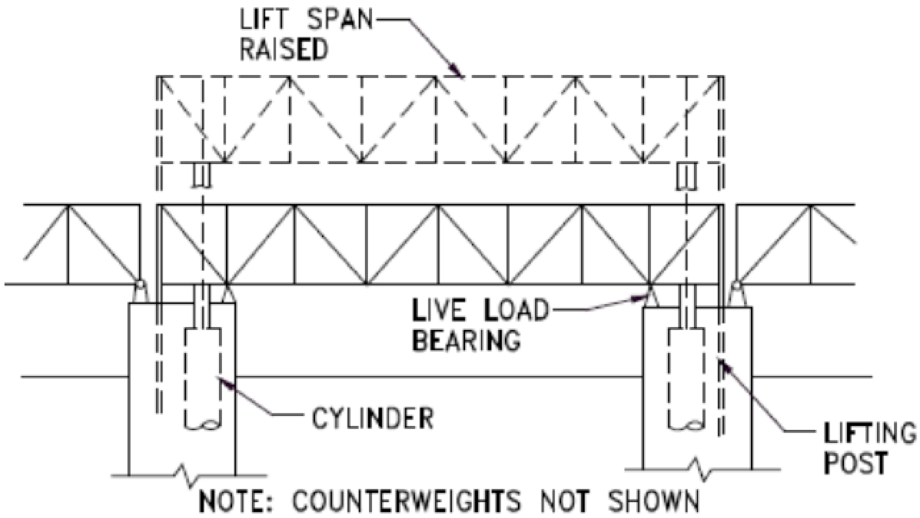


Fig. 2.22 - Pit drive vertical lift system (not to scale) (Berger, Healy, & Tilley, 2015)

3

MOVABLE BRIDGE CONCEPTUAL DESIGN

3.1. GENERAL

The design process of a bridge can be divided into three major stages: conceptual design, preliminary design and detailed design. The lifecycle comprises six main topics as shown in Figure 3.1.

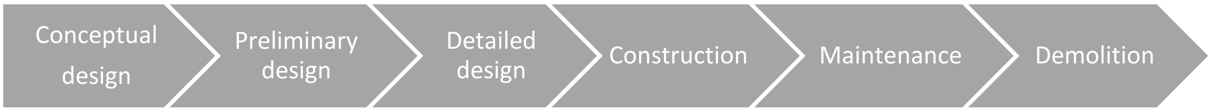


Fig. 3.1 - Main bridge lifecycle

Conceptual design

The concept design starts with the Brief of the owner/client – highways agency or regional or local authority – where the location of the bridge and main objective are presented. It is at this stage that the understanding of surrounding features, and possible constraints is critical. Aspects such as economic, social and cultural impact are particularly relevant and have to be taken into account.

The overall process goes on with the set-up of various road alignments from the highways team alongside possible bridge configurations from the structures/bridge team. For the bridge itself, this is the stage where certain features are considered, such as: number of spans; type of articulation; load path; and thus the type of bridge – truss, arch, beam and slab, box girder, etc. Other disciplines teams also take part in the process of the initial selection such as: environmental, transport planning and geotechnical.

After high level discussions between all disciplines and the client, the number of initial options – ranking usually from 10 to 20 high level options – is typically narrowed to 2 - 3 road alignments. Following this, different types of bridges are proposed accounting for the aforementioned features and constraints, where a number of 3 to 6 concept options is presented to the authorities responsible for the decision making development.

Preliminary Design

The preliminary design stage follows the decision on 1 of the options of the previous stage – conceptual design – and depending on client's requirements it defines the nearly final configuration of the bridge. The type of foundation is selected and pre-designed based on further detailed information about the ground conditions. The overall geometry is designed based on fair estimates of the structural performance of the whole bridge and of the main structural elements, which is usually carried out with some finite element modelling support for calculations. The overall configuration of the bridge should be defined to be used in the detailed design stage without significant changes. Cost and duration of the construction phase should be able to be estimated with good level of accuracy.

Detailed design

As the title of this stage describes, all that regards the final bridge definition needs to be detailed. Design calculations should be as sophisticated as possible. Written specifications are also required with lengthy description of construction materials and systems along with method statements of specific construction activities directly related to the structural performance of the bridge, given the various construction stages. Drawings are required to cover all the details for the accurate definition of all structural elements and those related to the articulation – expansion joints and bearings – as well as stringcourses, parapets, protection fences or any existing cladding to parts of the superstructure, piers/columns or abutments. In detailed design drawings are the most significant output of this stage. They are the main description of how the bridge has to be constructed from the location definition to the tiniest detail of a connection bolt.

The whole design process requires thorough understanding of the impact of type of loading, ground condition, structural behaviour, material properties and maintenance and construction concerns. Additionally, throughout all stages liaising with various other disciplines is essential to frame the conception and design of the bridge in the surrounding environment such as: Transport planning, Highways, Environmental/Social impact and in this particularly case Mechanical & Electrical Engineering.

A strong design that is based on a good concept and a continuous evaluation process leads to a more successful final solution than a design that is based on a refined concept. This is also responsible for cost and time savings in the long term of the project, as it can be seen in Figure 3.2.

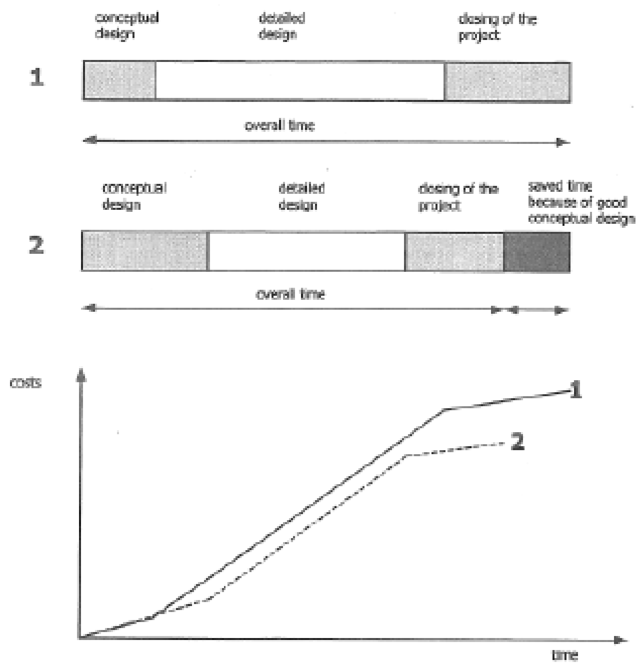


Fig. 3.2 - Potential savings in during a project (Nedev & Khan, 2011)

If the needs are correctly identified at the beginning of the project (conceptual phase) then the risk of changes in later phases of the design can be reduced or even eliminated.

In conclusion, the conceptual design is the key to ensure an optimum solution regarding all the issues and demands that are going to be highlighted afterwards.

3.2. METHODOLOGY

It is considered that a good approach to a problem is the basis for a good solution. Although the essential results may turn out to be the same, the approaches addressing the conceptual design are different for each designer. The main problem in the conceptual approach is to understand the process and the interconnecting needs that are not always obvious from its initial brief.

After some research, several of the main steps that were created are presented here for a successful method of designing movable bridges.

Step 1 – Project Definition

- Identify needs
- Identify problems and challenges
- Opportunities and constrains
- Background
- Future changes

Step 2 – Identification

- Navigation information
- Traffic information
- Topology condition
- Location considerations

Step 3 – Design Development

- Structural design demands
- Select the suitable components
- Search for solution principles
- Combine and firm up into a range of different concepts

Step 4 – Design Refinement

- Refine and improve performance of the different concepts selected
- Cost estimations
- Drawings and documents

Step 5 – Evaluation

- The proposed solutions/concepts are evaluated and ranked according to different parameters
- Sufficient – Choice of the final solution to further appraisal
- Insufficient – Go back to step 3

Fig. 3.3 - Process for conceptual design

The proposed methodology for the conceptual design of movable bridges can be seen in Figure 3.3. The output of this procedure highlights which concepts or demands are the most suitable for a design case. This approach is divided in five main levels with sublevels and is described in detail below:

First of all, the start of the project. The main interaction at this point is between the owner's objectives and the design team. It is important to identify all the needs and challenges that come up within the new project. A study of the background and future changes of the location has to be carried out to detect the different opportunities that can be achieved and the constrains that will impose the criteria and feasibility of the project, being of economic, environmental, social, historical and cultural aspects. It is also very important to recognise the desires of the society concerning a new bridge. Everyone that makes part of the project has to be aware of the main goals, objectives, problems and procedures that have been establish in this step and will be along the design.

Secondly, in this phase there is a continuation of the study and identification done in step 1 but more detailed to the specific site conditions or constraints. It is carried out the navigation study, along with a traffic study as well. It is a key point not to make the mistake of thinking not on a concept but on a specific solution straight away. This immediately narrows and restricts engineers view, limitation the possibility of an innovative design. After identification of the needs, they have to be analysed, which helps to set the limitations of the project and it is here that a refinement of the owner’s desires and all the needs and constrains find within the project.

It is noted that this two steps can be placed together in one, because the objective is the same, that is, to gather information for the definition of the project. But it was choose not to do this, in order to highlight the navigation study that is one of the most important demands in the development of a conceptual movable bridge design where the concept will start to take form.

The design development step is the start of the ideas and solutions process. A variety of solutions will be developed based on the design demands (described in the next chapter). A simplification of the process is done, removing the factors that are not important in the beginning or during the conceptual design phase but can be relevant in later phases. The gathered information is selected and at the end of this stage three to six different solutions are usually presented and analysed for public consultation.

The main goal of this stage is to refine and to improve the performance of all the different concepts providing more detailed information with preliminary calculations, sketches and documents – photo montage renderings, physical models, computer-generated videos, large presentation boards of the options, design quality plan and elevation drawings. This process requires more rigorous technical consideration such as preliminary loadings, vessel collision forces and evaluation of the structural analysis configuration like dimensions and material choice. It is also usually carried out a cost estimations.

The final step corresponds to the selection of the final option through the various design parameters that will move next to the preliminary and detailed design and ultimately construction.

3.3. DEMANDS

For a good bridge design, it is necessary to follow a wide range of demands in step 3. A list of these demands is set for the design of bridges and so adapted for movable bridges in the present work. See Table 3.1

Table 3.1 – Design demands

Demands	**Technical**	• Resistance • Serviceability • Safety • Codes and Standards
	Aesthetics	• Functionality • Proportionality • Accessibility • Integration into the Environment • Appearance
	Environmental	• Sustainability • Effect on the nature and resources
	Buildability	• Construction sequence • Construction time • Health and Safety • Building technology
	Economical	• Construction Cost • Maintenance Cost • Cost-Time-Quality
	Service life	• Durability • Maintenance • Inspections

Technical demands

Being the basis of any structural project, technical demands are very important and are normally restricted and described in codes and specifications. This has to satisfy resistance and serviceability requirements, i.e. the bridge has to resist all imposed actions and without collapsing (stability in all directions) or excessive deflections and vibrations (accommodation of movement). It is also a fundamental requirement the safety of the users of the bridge with a traffic and navigation safety layout and an easy maintenance.

Aesthetics demands

The aesthetics have an important role and should not be put to second plan because of economic or technical demands. This is because the public is getting more aware of the appearance of bridges and their effects in their daily life.

Every single element of a bridge has a visual design that have to be considered. The major elements that tend to be the most identified are: line, shape, form, colour and texture.

For the success of a good bridge design in the last and present century, aesthetics have to be considered from the beginning and the following criteria must be accomplish: simplicity, good proportions, functionality and integration with the surroundings.

Environmental demands

This matter only started to gain more significance in the last decades with the care that population is putting towards pollution and natural resources. Construction plays a big role in the environment impact.

Buildability demands

Buildability demands include the location, constructive method, transportation, time of construction and fabrication. These are all interconnected and have to be established along the process of design because it restricts the behaviour of the bridge (strength, deflection and stability).

Movable bridges should be designed in a particular sequence of construction in order to allow the stresses defined by the designer. Therefore, the fabrication and erection of the bridge has to be establish in a way that not interfere with this matter.

It is required to take into account the location of the new bridge and the transportation method possible for that location and design. For that, local restrains have to be establish, such us availability of skilled labour and equipped contractors and access and storage of the site.

Last, and maybe one of the most important aspect is the construction time has a big influence on the choice of the material and structural scheme as for movable bridges this interfere with the navigation traffic.

Economical demands

Of all the demands considerations, probably the most important one is the cost of the whole bridge project.

More and more the whole-life costs of the design are a big concern for engineers to reach a right balance between capital cost and operational/maintenance costs and a suitable approach to meeting the demand Cost-Time-Quality.

Different level of cost estimates detailing are made at different stages of the project. In the early stage an overall cost estimate (capital construction cost) is carried out commonly based on previous experience of similar projects and in the later stages is done a detailed cost which associates with risks and uncertainties.

Service life demands

The need to predict service life components of the bridge under different exposure conditions and constrains and the use of suitable materials and details according to their location is a key point of durability and consequently a good service life of the bridge. It is also required a proper plan for maintenance, inspection, rehabilitation and replacement of all elements of the bridge.

All demands are at some point interconnected, so a good design project depends on the ability of the designer to efficiently develop the best technical design without prejudicing other design demands.

There are two point of views remarking for the success or failure of a movable bridge design in the eye of the public remains in some aspects:

- Appearance
- Functionality

The absence of the cost is commonly set aside by the public in comparison with the others. Though in the eye of the owner/designer the most important aspect lies on:

- Costs
- Functionality

The functionality of the bridge by means of construction and maintenance costs considerations is what matters the most, so if the owner is not interested in an 'iconic' type of movable bridge, it is common to forsaken the aesthetics feature.

3.4. Structural Design

3.4.1. Design Criteria

The design of movable bridges requires much more effort than for the design of fixed bridges, for the reason that it must be taken into account the various position configurations of the leaf bridges and corresponding loads changes. So it has to be considered two different approaches, when in closed position, which movable bridges are designed for the same design conditions and procedures as fixed bridges, and when in open position, which are designed following some specific conditions:

- inertia forces of the moving span due to acceleration and deceleration during the operation;
- frictional resistance of the machinery;
- malfunction and failure of the electro-mechanical devices;
- Impact of vessel.

In addition, there is a number of elements details and issues that have to be considered, such as the interaction of the structure and machinery, like locks, bearings and others. These will be detailed along the present chapter.

3.4.2. Standards and Specifications

Apart from the adapted codes from the Dutch (Nederlands Normaisatie Instituut, NEN) and the Germans (Deutsches Institute fur Normung, DIN), currently there are no specifications for movable bridge design outside the United States of America, in english (AASHTO standards and AREMA). Although is commonly good practice to use the Eurocodes and/or design standards, these are incomplete in the detailed issues regarding the mechanical and electrical design. (Birnstiel, Bowden, & Foerster, 2015)

Hence it is up to every project owner to commend if the necessity for such codes is needed.

3.4.3. Constructive Materials

Structural and mechanisms material properties comprise one of the key points that have to be carefully considered because these are directly connected with durability and safety of the bridge.

In the past, the most common materials used for structural elements in movable bridges were wrought iron and steel. In the past few years some innovative movable bridges have been developed with aluminium and FRP (fibre-reinforced polymers).

The main aspects that the materials are required to have are:

- Hardness
- Fracture
- Tensile properties
- Residual stresses
- Corrosion
- Hydrogen embrittlement

Different materials are used for structural and mechanical elements and are going to be described in the respective chapters, though that sometimes they can be the same.

3.4.4. Balance

Regardless of the type of movable bridge, the balance of the bridge is a key issue to be addressed. Typically, to answer this problem, all vertical lift and bascule bridges are counterweighted. This is because it can minimize the size and power requirement for the machinery to manoeuvre the bridge, only needing to overcome inertia, friction, wind loads, imbalances and ice loads if it is the case.

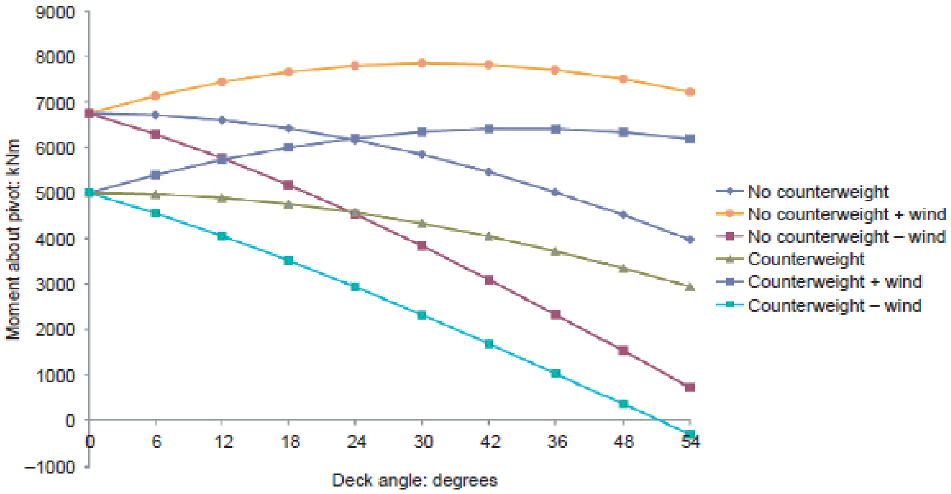


Fig. 3.4 - Graph of moment about pivot compared with angle of deck inclination (Thorogood, 2011)

It should be known that the wind loads can change to a direction that helps the lifting of the bridge bringing the moment close to zero with 50 degrees inclinations (see Figure 3.4). This brings a load reversal (from compressive to tensile load) in the lifting mechanism. Although not a major problem, this can pose some issues to the control and support of the bridge in closed position which has to be considered in the design stage.

It has to be noted as well, that the counterweight has to be designed to allow for adjustment of the bridge balance, as with time the weight and weight distribution can change, due to repairs, paint, replacement of locks, etc. This can be supported by having balance checks during construction or using detailed calculations comprising every structural element as well as coatings and paintings that contribute to the weight of the moving span. For more detailed balance calculations see the paper of Giernacky and Tosolt (2010).

There are primary values that can be used in the early stage of the design, which for vertical lift bridges is every kilogram of the total weight is balanced by a kilogram of the counterweight, 1:1 racio. And for bascule bridges the counterweights normally weight three to four times more than the weight of the movable span.

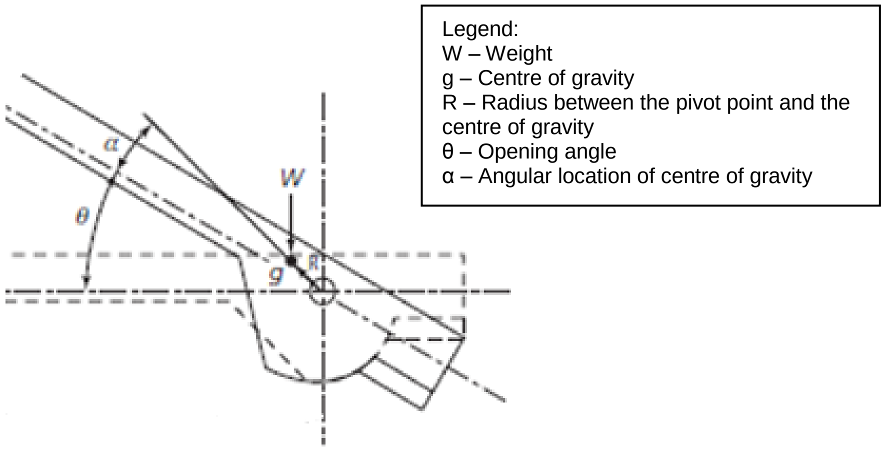


Fig. 3.5 - Balance bascule leaf (adapted Birnstiel, Bowden, & Foerster, 2015)

Note: Valid for trunnion and rolling bascules, for more information about other types see (Hool & Kinne, 1943)

There are various views about balancing a movable bridge. Balancing bascule bridges is particularly more difficult as it has to consider both the vertical and horizontal location of the centre of gravity of the leaf. It is, commonly, considered a slightly "span-heavy" condition to create a tendency for the moving span to stay in closed position without needing machinery. This corresponds to a centre of gravity towards the navigation channel. For vertical lift bridges this is easier, only needing to consider the horizontal location of the centre of gravity of the leaf and a slightly overall heavy span. (Coates & Bluni, 2004)

Hence, the moment needed to overcome imbalance, M_b, is given by the fundamental balance equation:

$$M_b = W.R.\cos(\theta + \alpha) \tag{3.1}$$

A "span-heavy" condition is given when a positive value of M_b, i.e. the moment necessary to open the leaf in this condition. With this, usually is applied a force to the toe of the leaf (termed Toe Reaction, T_r) that is equivalent to the imbalance moment:

$$T_r = M_b.L \tag{3.2}$$

It is exemplified, afterwards, by means of movable bridge balance tests, the different cases of torque and consequently balance/imbalance of the bascule bridge in Merritt Island, FL. (see work paper Susoy, Zaurin & Catbas, 2007)

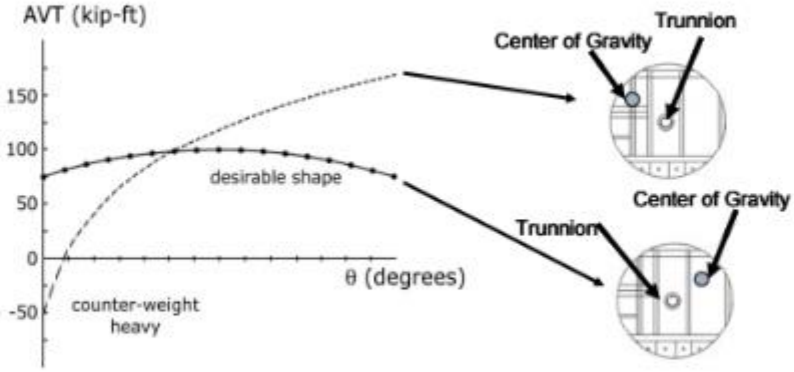


Fig. 3.6 – Comparison of different torque during operation of a bascule bridge (Susoy, Zaurin, & Catbas, 2007)

Comparison between the different motions of two bascule bridges is demonstrate below:

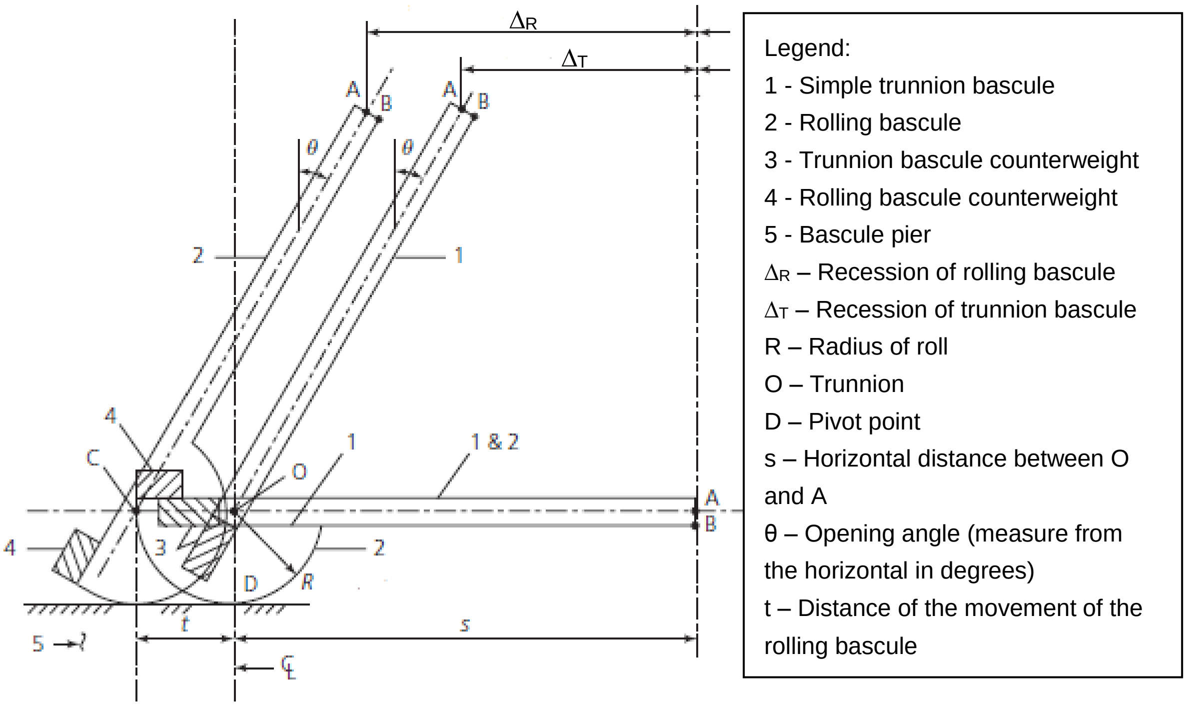


Fig. 3.7 - Comparison motions between trunnion and rolling bascules bridges (adapted Birnstiel, Bowden, & Foerster, 2015)

It can be seen thru figure 3.7 that the trajectories of the movement of bridge opening of the trunnion and a rolling bascule bridges are different. Though the position in closed position is the same, when it starts to rotate about the same opening angle, the bridges rotates from a different motion. The trunnion rotates through a fixed point (pivot point called trunnion) proving a recession inferior to that of a rolling bascule that rotates through a rolling track.

This proves to be a very important feature when it comes to navigation channel with height ship traffic, attesting that a rolling bascule (Scherzer) is a better option for this cases.

3.4.5. Superstructure Forms

As described in the chapter 2, there are many types of superstructure forms that were developed over the years with different design approaches in mind. In the 19^{th} and 20^{th} centuries the design of movable bridges was dominated by solid structures, with an attention for first and operating costs, disregarding aesthetics. This created an overall discontent of the public that wanted designs more ‘iconic’.

In recent years the consideration of aesthetics went to first plan and some concepts that were considered for being practical are being put aside. Though the concept of beautiful changes with time, now the guidelines for design of movable bridges requires factors like clear structural lines, good proportions and above all simplicity. Nevertheless with movable bridges it is never easy this line of thought because a lot more key aspects have to be considered than for fixed bridge, such as the interaction of reliable operation with reasonable costs and location.

3.4.5.1. Geometry VS Materials

One of the most important matters in movable bridge decks is the reduction of self-weight, because not only it is easy for operation purposes of the lift machinery, as it gives more support capacity for live and seismic loads.

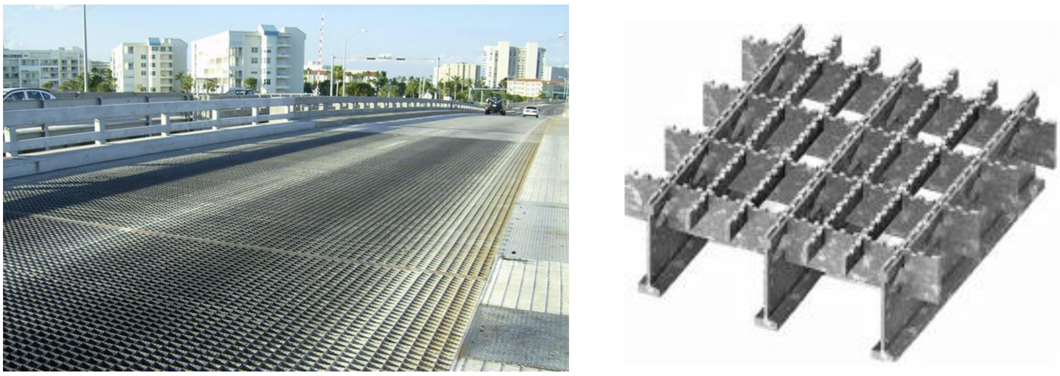
When it comes to choose the right kind of deck form, various matters have to be considered regarding:

- Bridge settings (locality, water level)
- Buildability
- Traffic Load
- Type of maintenance required
- Costs

The most common in the present days are described below:

Steel grid deck

- **Open grid deck**

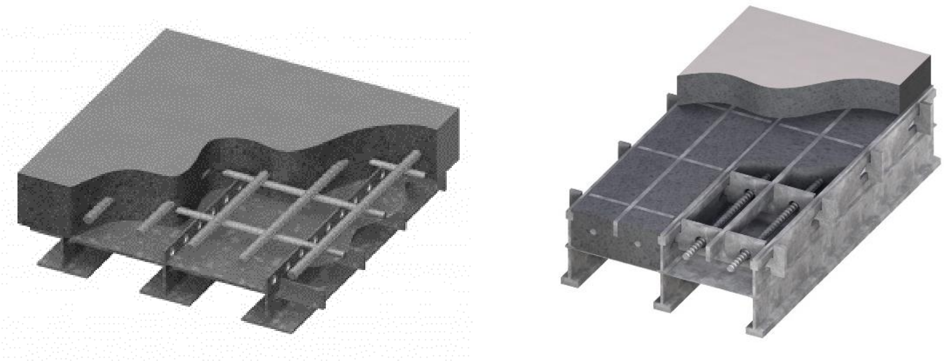


(a) (b)

Fig. 3.8 – (a) Bridge Corey Causeway (Unknown, DrawBridgeAhead.com, n.d.) (b) Rectangular open grid (Birnstiel, Bowden, & Foerster, 2015)

Open grid steel decks have the lightest weight of all type of decks and particularly for bascule bridges reduces exposed wind area significantly when in open position. However, it increases the number of accidents with traffic high speeds and congestion, it is noisy and it can accumulate debris and rain water which leads to corrosion.

- **Exodermic and Concrete filled grid deck**

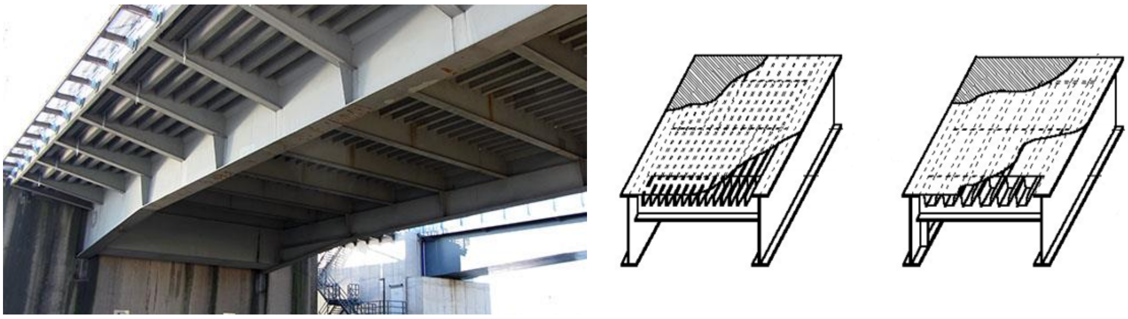


(a) (b)

Fig. 3.9 - (a) Exodermic deck; (b) Half-filled concrete grid deck (Birnstiel, Bowden, & Foerster, 2015)

Exodermic and concrete filled grid decks (full or partial filled) have the advantage of not needing welding to supports and the possibility of construct larger spans.

Orthotropic steel deck



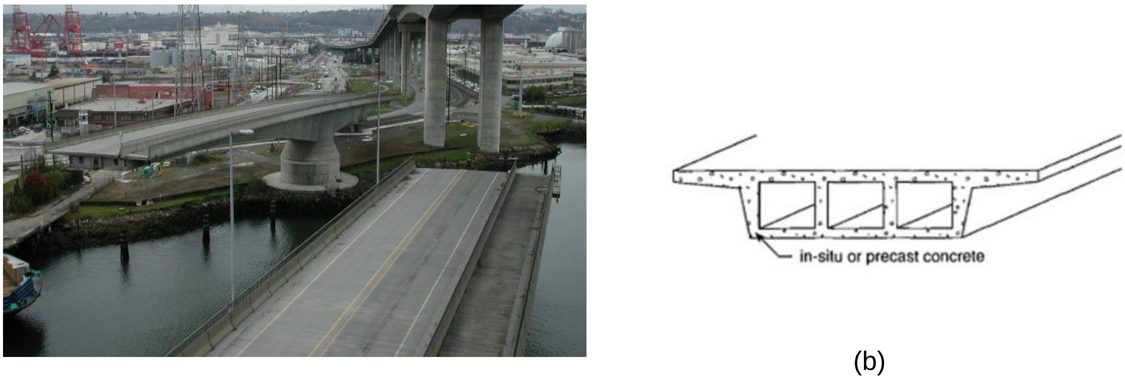
(a) (b)

Fig. 3.10 – (a) Movable bridge across Hartelkanaal (Royal HaskoningDHV, 2014) (b) Orthotropic plate decks (Unknown, ESDEP Course)

Orthotropic steel deck gives a light weighted solution which has a beneficial interact with the remainder of the bridge structure. This causes a really good seismic performance.

The main issue that has to be overcome is the wearing surfacing of this deck types which has to be light weighted, durable and particularly in bascule bridges has to be adhesive due to the lifting operation. It is more expensive than open grid steel decks.

Concrete deck (Light-weight concrete)



(b)

(a)

Fig. 3.11 – (a) Spokane Street Swing Bridge (Yashinsky, 2010) (b) Concrete deck (O'Brien & Keogh, 1999)

When using concrete deck, it is required stainless steel reinforced bars and lightweight concrete to allow a minimum cover and hence control the deck weight.

Aluminium

(a)

(b)

Fig. 3.12 – (a) Bridge Han Lammersbrug (Koutsarsky, 2012) (b) Aluminium deck (Aluminium Association of Canada, n.d.)

Aluminium decks are recently being developed, as well as FRP decks, for being light weighted, durable, low in maintenance and having high fatigue strength.

3.4.5.2. Proportions

For purposes of conception and initial sizing, some basic “rules” are acknowledged for economic strategy and are summarized afterwards.

Double leaves swing bridges proportions in comparison with front and back span comprises usually a ratio of 1/2 of the total length. For single leaf swing bridges the tail or backspan are typically 30 – 40% of the main span. For bascule bridges usually around 1/3 of the total length but depends a lot on the counterweights layout and type of bascule bridge.

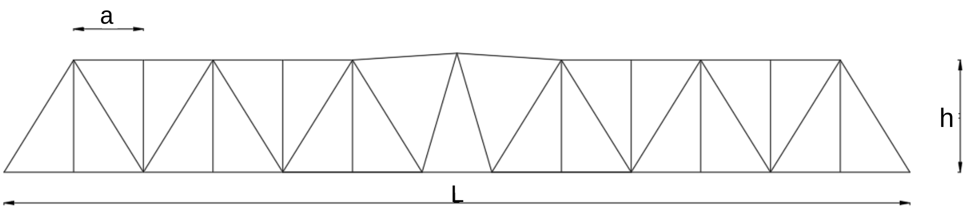


Fig. 3.13 – Initial sizing - truss (not at scale)

The section depth is typically between $\frac{a}{20}$ and $\frac{a}{25}$ in steel elements in general, being a the major element. The height of the truss is normally in the range of $\frac{\mathrm{L}}{11}$ ratio.

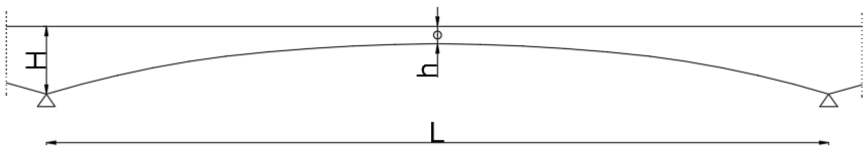


Fig. 3.14 - Initial sizing – haunch deck (not at scale)

When the section varies depth along its length, usually for bascule bridges, the typical ratio of the height of the support (H) is between $\frac{L}{16}$ and $\frac{L}{20}$ whereas in the mid-span (h) is between $\frac{L}{40}$ and $\frac{L}{50}$.

3.4.6. Structural Behaviour and Load Paths

The main structural members of the bridge conditions the entire structure due to their own weight and aerodynamic unique characteristics. A good understanding of the load paths is needed, as well as the behavior of support conditions and the interaction between the different moving elements of the movable bridge.

3.4.6.1. Bascule Bridge

The main structural members of bascule bridges can be either trusses or girders. For purposes of simplified analysis of the structural behaviour and load paths of bascule bridges, it is only considered trunnion bascule bridges, as these are the most general case. For a detailed analysis see Hool and Kinne, 1943.

Rotation of the span is supported by the drive mechanism and when it comes up to the open position the span weight is supported by the trunnions. These trunnions are subjected to lateral loads which are resisted by the piers. In the leaf these lateral loads should be resisted by a lateral bracing system.

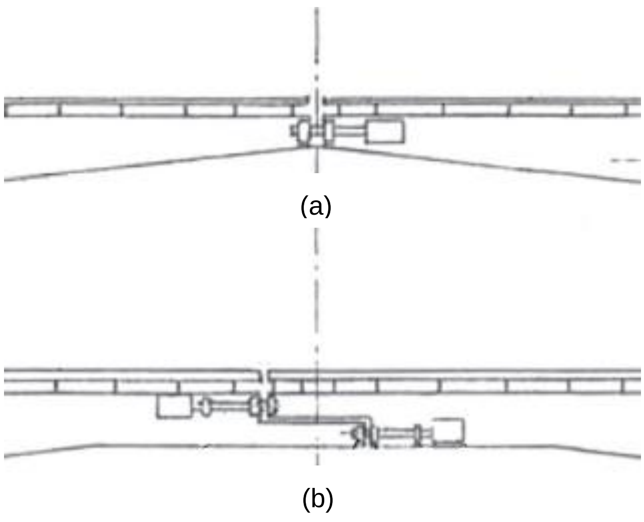


Fig. 3.15 - Shear and moment locks arrangements: (a) shear lock; (b) moment lock (adapted Birnstiel, Bowden, & Foerster, 2015

When in closed position, the centre locks and devices usually only offer limited lateral resistance and nearly no longitudinal resistance. However, this can be improved with the application of tail locks which transfer the longitudinal resistance to the trunnions. For the structural analysis of these bridges it has to be knowledgeable the behavior of this distinctive points and how it affects the overall structure. When modeling single leaf bascule bridges the behavior rolls up to a cantilever for dead loads and a simple span for live loads. In the case of double leaf bascule bridges modeling, the behavior rolls up to a cantilever for dead loads and for live loads depends on the type of connection made in the junction of the leaves.

If the connection is made with shear locks which transfer shear forces, the girder behaves like an elastic propped cantilever and if is made with moment locks, that transfer both shear and bending moments, it behaves like a continuous girder, theoretically, because the deflections on the mid span reduce the effectiveness of the system.

Fig. 3.16 - Schematization of real behavior double leaf Bascule Bridge with active mid-span shear locks

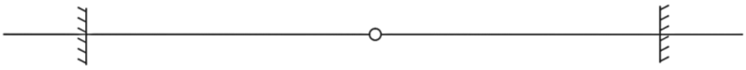


Fig. 3.17 - Analogy schematization of double leaf Bascule Bridge with active mid-span shear locks

The usual analogy that is considered for conservatism reasons when modeling the leaves for the shear lock system in a double leaf bascule is a hinged point. This because it provides the ability of transferring shear loads and it allows a leaf-tip rotation at the connecting point and expansion and contraction between the leaves due to temperature effects. This can be simplified by modeling only half of the bascule bridge considering k as the stiffness of the spring as much as the bending stiffness of the adjacent cantilever leaf.

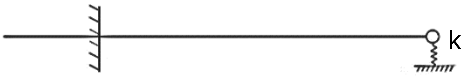


Fig. 3.18 - Simplified schematization of double leaf Bascule Bridge with active mid-span shear locks

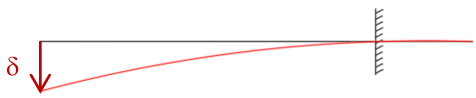


Fig. 3.19 – Displacement of adjacent cantilever leaf

$$\delta_{max} = \frac{P.l^3}{3.EI} \tag{3.3}$$

$$k = \frac{P}{\delta max} = \frac{3\,EI}{l^3} \tag{3.4}$$

It has to be noted that this type of lock system is not suitable for heavy rail traffic due to the sudden change of profile so it is usually employed moment locks.

The usual analogy that is considered when modeling the leaves for the moment lock system in double leaf bascule bridges is two point supports.

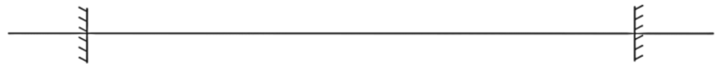


Fig. 3.20 - Analogy schematization of double leaf Bascule Bridge with active mid-span moment locks

Moment locks can reduce the expected deflections by more than half when compared with shear locks.

3.4.6.2. Swing Bridge

The main structural members of swing bridges can be also either trusses or girders. As the structural behaviour of the trusses or girders changes among the different positions of the bridge moving spans, the deflected shapes due to the self-weight are different. So the analysis of swing bridges has to consider the following requirements for both the superstructure and machinery (discussed in section 3.4.7). It is considered in the present study the load paths for double-arm swing bridges comprising trusses.

In open position, the weight of the span is supported by the centre bearing in centre bearing swing bridges, and by rollers in rim bearing swing bridges. In both cases the swing spans work as a double cantilever, balanced on the pivot point. On the process of opening the bride, tilting can occur so it must be resisted by the balance wheels assembly in centre bearing bridges and by the rim bearing assembly in rim bearing bridges.

In closed position, the swing span is supported by three points in centre bearing swing bridges, the centre bearing point and two rest piers, one at each side, and by four points in rim bearing swing bridges, two points in the rim bearing assembly and two rest piers.

Swing bridges can have various structural forms depending on the designer choice as long as the stiffness and strength required for this type of bridges is achieved.

For purposes of a stiffer span and to restrain the compression chord, the common layout is the through truss which provides bracing between the two upper chords. Horizontal loading is transferred through the lateral bracing system.

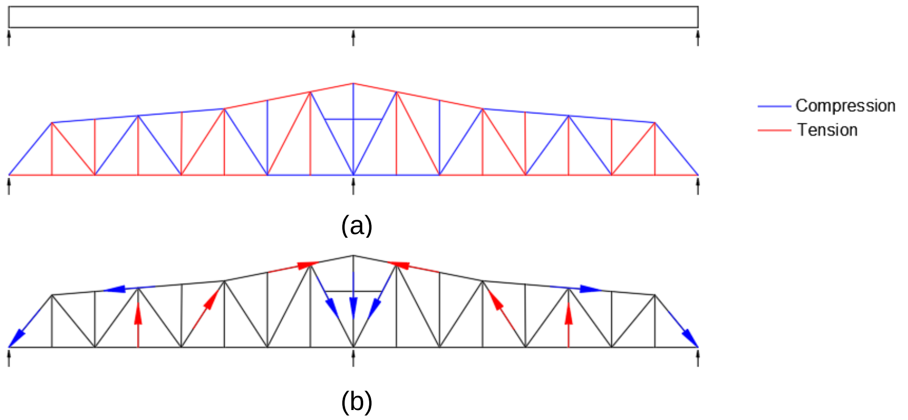


Fig. 3.21 - (a) Stresses diagram of centre bearing Swing bridge (b) Load Paths of centre bearing Swing Bridge closed position (not at scale)

In open position the stress in the bottom chords, close to the middle of the arm, due to dead load is compression and in closed position this stress is tension. Normally near the centre support there is a truss arrangement more detailed with the purpose of not transmitting much vertical shear and a reinforced bracing since this space is subjected to greater forces.

This arrangement helps to uniform the loads going to the pivot pier, proving it to be the best solution for minimising the possibility of uplift of the one of the centre points due to live loads placed on the opposing span.

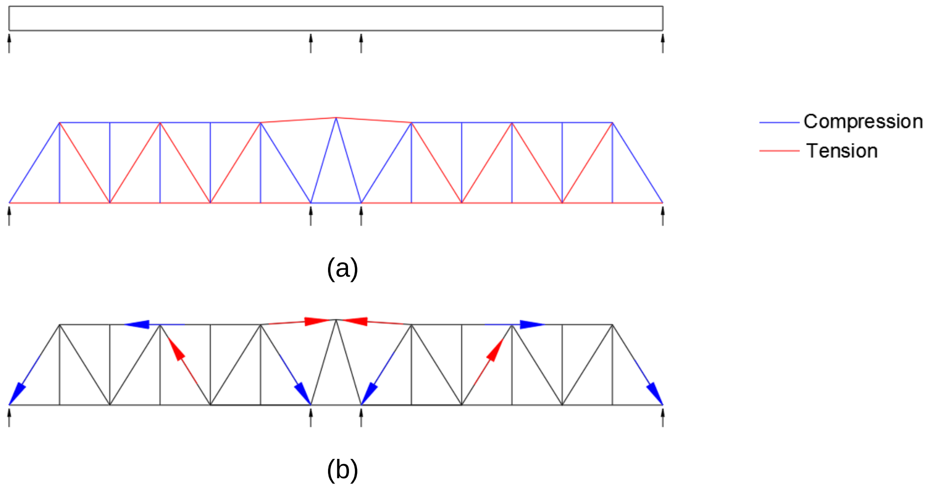


Fig. 3.22 - (a) Stresses diagram of rim bearing Swing bridge (b) Load Paths of rim bearing Swing Bridge closed position (not at scale)

In symmetrical swing bridges, the ends are lifted (with devices) when the rotation for closing the bridge is completed, providing a preload reaction into the swing span. This has the purpose of supporting additional traffic loads and avoid the uplift due to the case when the swing span is only loaded in the opposite side.

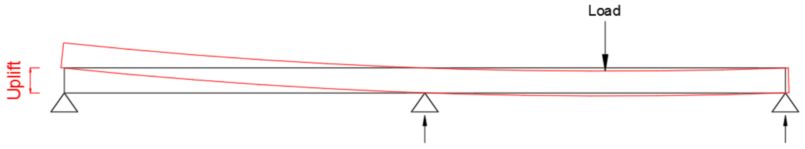
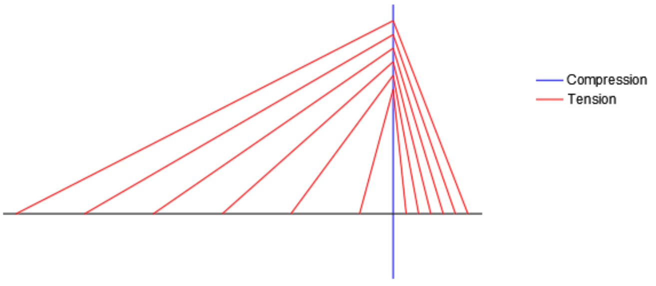


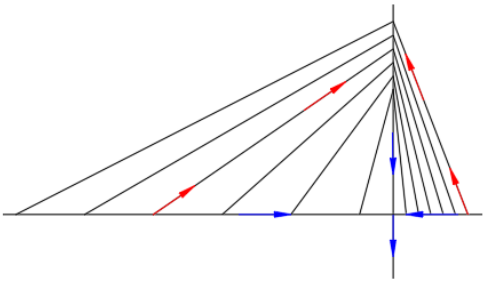
Fig. 3.23 - Deflection diagram of centre bearing swing bridge (not to scale)

In the case of cable-stayed swing bridges, the orthotropic deck is in compression and cables in tension. Cable-stayed swing bridges are basically the combination of two types of bridges: swing and cable-stayed bridges. This arrangement was with the purpose of reducing the bending moment of the main pier (termed pylon) base.

The moment depends on the stiffness of the deck which in turn depends on the cable arrangement and shape of the deck.



(a)



(b)

Fig. 3.24 - (a) Stresses diagram of Cable-stayed Swing bridge (b) Load Paths of Cable-stayed Swing Bridge (not at scale)

3.4.6.3. Vertical Lift Bridge

Again, the main structural members of the span of vertical lift bridges can be either trusses or girders and the main structural members of the lift towers can be trusses or reinforced concrete.

The weight and both longitudinal and lateral loadings of the moving span are all supported by the lift towers and counterweights system. It is imperative that the towers offers a reasonable resistance so it can withstand all forces.

The movement of each point where the span connects to the lift tower has to be synchronised in order to reduce the kinematic system to only one global movement, avoiding sway motions of the deck. This is the most important aspect to consider in designing a vertical lift bridge. To answer this problem, it is provided alignment and guidance devices for the lift span, counterweights and drive mechanisms (see section 3.4.7).

There are some other aspects that need to be considered in motion analysis, in different positions of the bridge. When the bridge is in open position:

- High shear, torsion and bending moment on the towers provided from lateral reactions from the lift span and counterweights;
- The base tower connection resists the tower loading by means of anchor bolts.

When the bridge is in closed position:

- The ends of the span are restrained by span locks and guides, and centering devices as it will be described in detail in the next chapter;
- The moving span has to be well braced transversely to be able to transfer the loads that are subjected to the towers.

3.4.7. Machinery Systems

All movable bridges requires some type of machinery for the purpose of a good performance and stabilisation of the bridge. Within this, their function can be from supporting dead and live loads, transferring shear loads or allowing and stabilising the operation of the bridge by locking and unlocking the movable span.

Various concerns arise when it comes to choosing this devices:

- Type of moving structure;
- Location within the structure;
- Type of operation device;
- Maintenance and inspection.

There are different types of machinery devices according to the different type of movable bridge and their location depends on the structural requirements for support and/or restraint. It is important as well, to consider the local environment that the bridge is subjected together with the location of the devices, for purposes of maintenance and inspection of the bridge.

It will be considered here the most common types of machinery used according to the three main types of movable bridges.

3.4.7.1. Bascule Bridge

- **Live Load bearings**

There are two variations of live load bearings for double leaf bascule bridges:

Forward live load bearing or Live load shoes – these are located between the trunnion or tracks and the main pier next to the water, underneath the bascule girder and provides a reaction point in front of the trunnion. This bearing point allows the reducing of the trunnion or track loading by reducing the moment arm of the live load traffic. These supports are rather durable, only being subjected to deterioration by means of corrosion.

When the bascule bridge is of a single leaf this bearing situates at the toe of the leaf as showed in Figure 3.26.

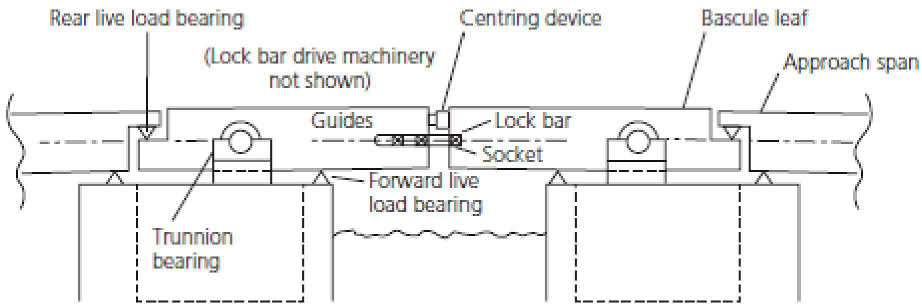


Fig. 3.25 - Illustration of the majority of machinery for double leaf bascule types (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

Rear live load bearings or Live load anchorages – these are located near the counterweights and provides a reaction point behind the trunnion. This bearing brings the centre of rotation approximately to the wall close to the river, allowing a construction of a smaller pier. These have the disadvantage of creating a higher trunnion loading and span moments. In addiction has a difficult access so are more subject to misalignment and consequently instability.

- **Span locks**

In double leaf bascule bridges these span locks are also called centre locks as they join at mid span. Their purpose is to transmit shear force or bending moments between the leaves, depending on the type of lock system. This, forces the leaves to share the load and have the same deflection, allowing longitudinal movements caused by temperature. It also makes the leaves aligned and secure in closed position.

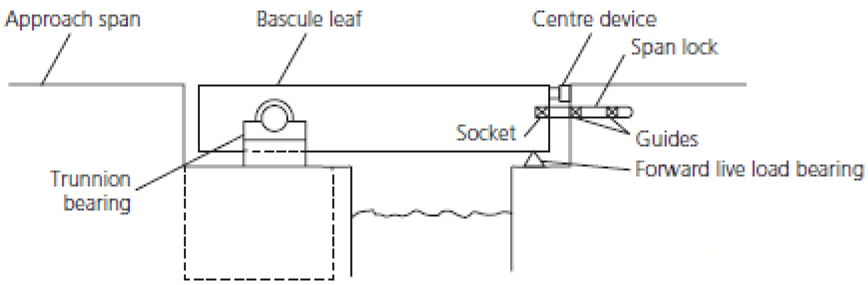


Fig. 3.26 - Illustration of the majority of machinery for single leaf bascule types (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

For double leaf rolling bascule bridges the locking system used is a jaw and diaphragm allowing, like the shear locks, the transfers of shear loads between the leaves. This lock is used in this type of bridge only because their arrangement allows one leaf to hook at the other when it rolls into or out the closed position. It is very important to regularly maintain these locks because if it wears, the mutual deflection cease to exist and each leaf bounces as live load passes from one leaf to the other.

- **Tail locks**

These have the main purpose of locking the bridge in closed position and prevent the leaves to open when subjected to live loads. These locks are usually at difficult access to maintain and inspect, so they are usually much damaged and misalignment.

- **Centring devices**

For a correct alignment of the bridge in the transverse direction it is used centring devices.

- **Bumpers**

Bumpers are normally used when there is a need for a deceleration of the moving span when electrical controls are not used.

It is a very difficult task to obtain a correct alignment and consequently a good operation of all the elements working together. For a double leaf bascule bridge it is possible to have a maximum of eighteen support points: (Koglin & Colker, Stabilization of Double Leaf Bascules, 1995)

- 4 Trunnions or tracks
- 4 Forward live load bearings
- 4 Rear live load bearings
- 4 Tail locks
- 2 Span locks

3.4.7.2. Swing Bridge

- **Centre supports**

Centre supports purpose is to provide live load support when the bridge is in closed position and restrict the rotation of the bridge when subjected to traffic and wind loads. These supports are disengaged when the bridge needs to swing to an open position. A common arrangement of these supports is illustrated in Figures 3.27 and 3.28.

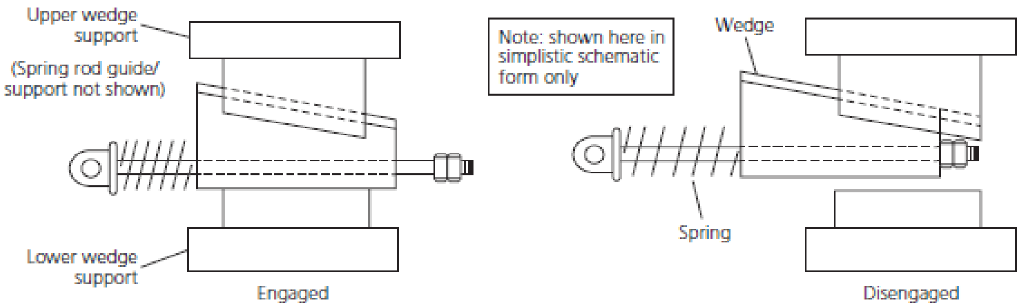


Fig. 3.27 - Illustration of the centre supports (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

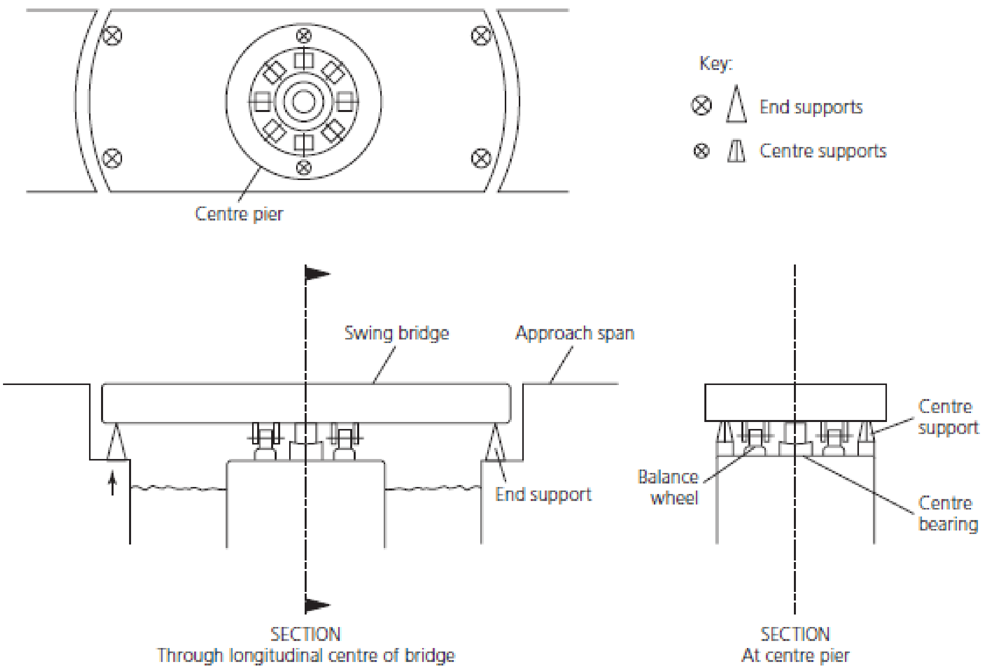


Fig. 3.28 - Illustration of the majority of machinery for swing bridges (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

- **End supports**

As well as centre supports, the end supports provides for live load support and restrict the bridge to rotate when in closed position. However, these have also a very important purpose, which is to prevent an uplift of the end leaves (described in section 3.4.6). There are various arrangements for this type of supports but one of the most common is showed in Figure 3.29.

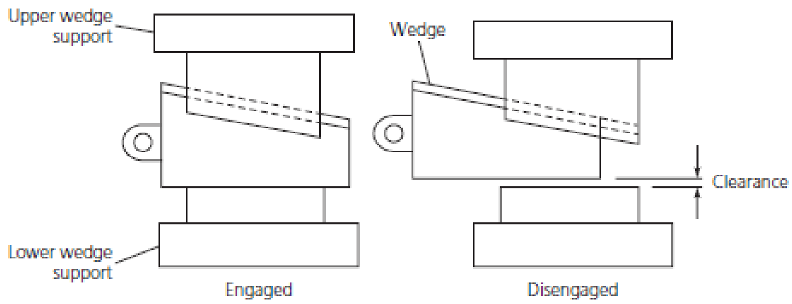


Fig. 3.29 - Illustration of one arrangement of the end supports (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

- **Balance wheels**

When the end and centre supports are disengaged to allow the swing span to rotate, it is needed some kind of support that helps to stabilize the span and prevent tilting from loads, like the wind. The design of this balance wheels consists in a minimum of eight wheel assemblies that roll on a large diameter (close to the width of the bridge) circular track, sitting on the centre pier.

- **Centring and locking devices**

In swing bridges these two devices can work separate or together as one device. These have the same purpose as described for bascule bridges, which is align and lock the leaves in their final position. They can be used as a horizontal bar mounted in the swing span or as a vertical bar with a roller that can slide up to secure the leaf.

Also, it is possible on swing bridges with end wedges to take advantage of this and use this devices for centring and locking the bridge. (See end wedges arrangements in Chapter 2)

- **Bumpers**

As for the bascule bridges, these type of devices serves the purpose of stopping or slow down the bridge. In this case, it is only used for swing bridges that do not rotate 360°. These bumpers are typically rigid devices to prevent any excessive displacement when the leaf stops.

3.4.7.3. Vertical Lift Bridge

- **Live load bearings**

These structural bearings provide the same aims as for bascule bridges but with the difference that they allow the end of the span to expand and contract. This is possible because one end of the span has fixed bearings and the other end expansion bearings. The expansion bearings are needed because vertical lift bridges have the feature of having a huge span length, which are subject to movements due to temperature changes.

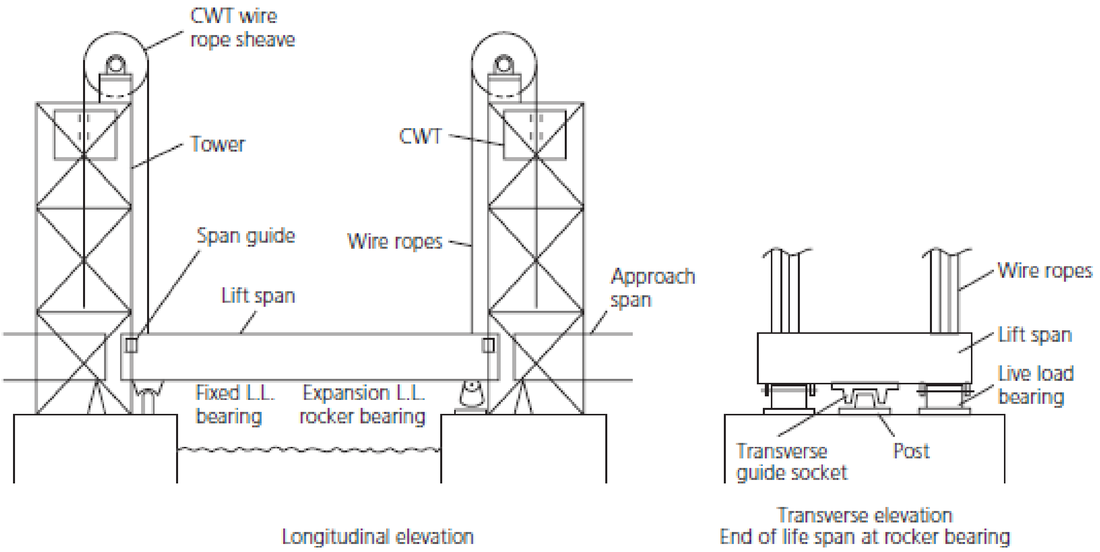


Fig. 3.30 - Illustration of the majority of machinery for vertical lift bridges (not to scale) (Birnstiel, Bowden, & Foerster, 2015)

- **Span locks**

Same purposes of span locks for bascule bridges.

- **Centring devices**

With the same purpose as for the other type of bridges, however is not used a system bar but a socket on the lift span and a post on the pier, as showed in figure 3.30. It is important that this device is designed to resist wind loads because when in closed position the span is seated on this device.

There are also some span guides devices that help set the correct position of the bridge. These have to be carefully detailed to be smooth to minimize friction.

- **Bumpers**

Same purposes of bumpers for bascule bridges.

3.5. OPERATING MECHANISM DESIGN

The operating mechanism system is part of the main key elements on which the entire design is carry out and it has to be given proper consideration to the type and location of the different equipment's (mechanical, electrical and hydraulic elements). The selection of the type of mechanism has to follow some aspects:

- Type of movable bridge;
- Available space;
- Proximity with water level and so the possibility of flooding;
- Skill base for maintenance and inspection.

Nevertheless, despite these aspects, owner preference and construction and maintenance cost continue to have an important weight in the final decision.

Currently the operation of movable bridges lays on either a mechanical mechanism or hydraulic with auxiliary mechanical machinery mechanism. In the mechanical type, every element is powered by 100% electrical power and commonly the drive system is a curved gear rack sited in the pivot. In the hydraulic, the drive usually takes form of cylinders or hydraulic motors.

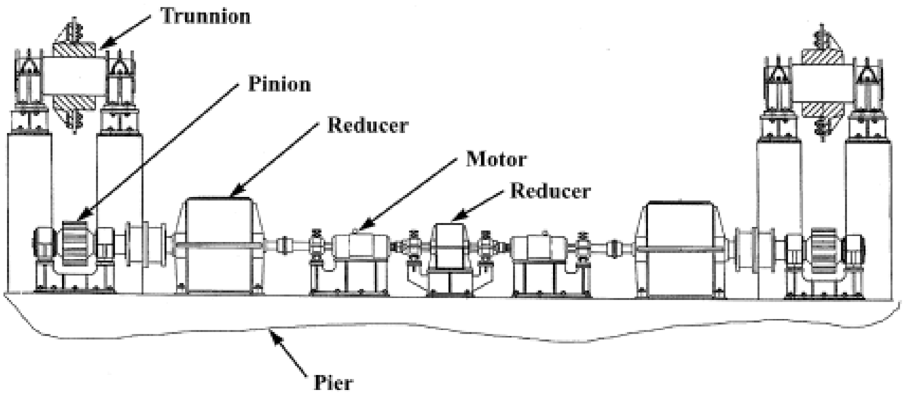


Fig. 3.31 - Section of a bascule bridge with a mechanical drive mechanism (Abrahams, 2000)

The advantages of the mechanical type of mechanism can be summarized in the following:

- The prime mover drive can be in the form of electrical motors utilising modern sold state electronics to control speed and equalise torque instead of requiring complicated hydraulic systems;
- The speed remains constant throughout the operating of the bridge except for the acceleration and deceleration phases;
- There are potentially no hydraulic pipes in the machinery chambers, thus less space is required and the costs are reduce;
- Larger opening angles are possible as the Kinematic moment arms remain constant.

This system requires regular inspections.

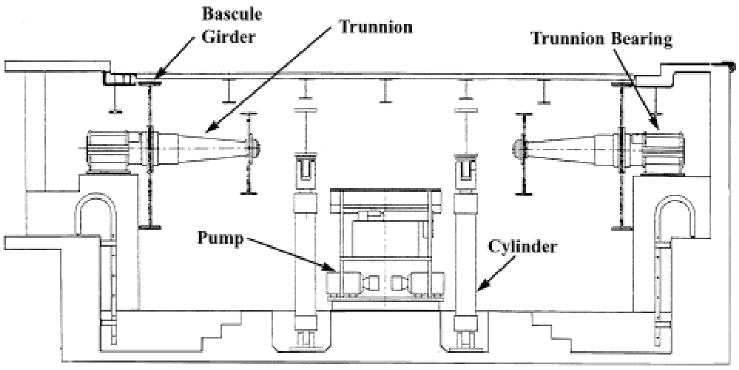


Fig. 3.32 - Section of a bascule bridge with hydraulic cylinders mechanism (Abrahams, 2000)

The advantages of the hydraulic type of mechanism can be summarized in:

- The load in the hydraulic cylinders in still air conditions remains sensibly constant, and no reversal of load takes place within them during the whole lift operation;
- Very robust system;
- Can share load across a structure using even the simplest hydraulic circuit;
- Less sensitive to water or flooding than a purely electrical solution.

The principal disadvantages are the need of specialised maintenance because is needed both knowledge about electrical and hydraulic machinery and the great environmental risk of oil spill (although very unlikely).

3.6. Safety Design

The greatest risk about a movable bridge structure is the potential hazard and danger that this can bring to the various users, being these, pedestrian, vehicle or navigational. Other than the safety of the structure itself, considered in the other sections of the present work, requirements for monitoring, operation, functional safety of the bridge and connection with the surroundings and the road connections are really key factors that must be considered as well.

3.6.1. Bridge Operation

Although it is thought that the time of lifting and closing the bridge is a key feature to a quick total operating time, this is not the case. The time that is needed to clear the pedestrian and road traffic dominates the total cycle time of the bridge. Hence, having a bridge that takes little time to open and close the span is of little benefit unless traffic control has also been optimised. And to achieve the reduced time for the operation requires a rapid acceleration and deceleration by means of much higher power of the lifting mechanism. This of course increases the cost not only of the mechanism but also of all the mechanical features.

There are two possible modes of operating a movable bridge. Both are operated from the Operator's control desk located in the control room and usually sited above the plant rooms, were it is provided good visibility of the bridge.

- Automatic – fully interlocked
- Manual Operation – step-by-step control

There is protocol for the bridge operation that has to be followed and a well-structured, powerful automation system with redundant control and network technology to ensure maximum availability of the movable bridge.

3.6.2. Traffic Control

All these equipment's described below have to follow specific and detailed requirements. These requirements classify the type of equipment and arrangements depending the type of bridge. However the project owner can impose additional requirements.

Table 3.2 – Main detailed requirements

Equipment	Description
Safety Gates and Barriers	- Safety gates comprises two types of gates: a warning gate and a physical barrier; - Each approach has to be provided with gates and extend across the full width of the bridge to stop access; - Powered electrically, however provision for manual operation is required, in case of malfunction or power cuts.
Traffic signals and signs	- Traffic signals have to situate close to each type of safety gates; - Two types of traffic signals can be used: three colour signals and multiple red signals in vertical array, according to traffic control regulations.
Navigation signals and signs	- Navigation signals and signs have to be sited on each side of the bridge spans and have to have suitable access; - Have to be in conform to the navigation authorities.
Lighting	- Red signal lights have to be provided on the safety gates and work simultaneity in all the operation; - Navigation lights have to be provided on each side of the bridge spans, requiring waterproofing and impact toughness.
Bells and Warning devices	- Bells and lights have to be provided along with the gates and work simultaneity in all the operation; - Warning bells or gongs have to be provided with the traffic signals according with the type of operation (manual or electronical).

There is a feature that needs more consideration which is the location of the gates and the distance between the bridge end and the barriers. The location of the gates must be right next to the gap where the bridge rotates or have a set back from this gap. Depending on the type of bridge and the surroundings next to the gap, like protection between the river/canal and the barrier, people are more susceptive or not to be close to these and to see the opening of the bridge.

The distance between the barrier and the end of the bridge is associated with the time necessary for the clearance of the road and pedestrian traffic. The further apart the barrier is the more distance to go through is needed and thus more time is required. Hence, two different sets of barriers are commonly used, one for pedestrians and one for road traffic. This allows a first clearing of the pedestrians and cyclists while prolonging the traffic flow before the start of the operation of the bridge, preventing congestion.

3.6.3. Ship Collision

Normally movable bridges are designed with the minimum allowable navigation channel for purposes of reducing the costs of elevating a bigger span. This and the development of larger and big vessels keep increasing the probability of a ship collision with movable bridges.

Apart from the consideration of a collision, which has to be undertaken in the design analysis of the bridge, it is required that a fendering system is constructed to protect the bridge, in both closed and opened position and in the different positions of the bridge, piers or the movable spans, or both at the same time.

When the fendering system suffers deformation because of ship impact, there should be no contact between the vessel and the pier or span. Special consideration must be included for the overhang of raked bows on ships and barges.

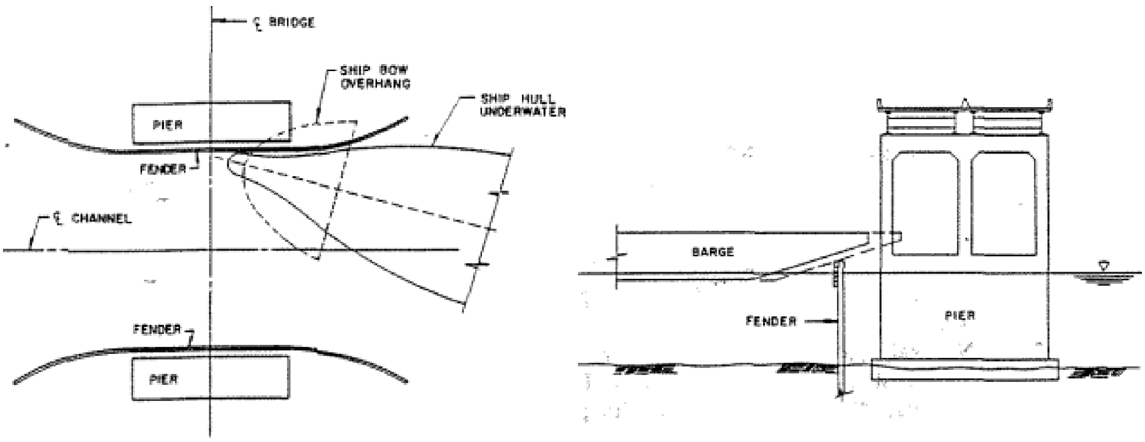


Fig. 3.33 - Plan of overhang and elevation of Barge Bow ship collision with pier (Knott, 1990)

3.7. Buildability

It is very important that the movable bridge design counts with the buildability of the bridge, i.e. the installation sequence and the availability of the operating equipment.

The manufacturing of all the elements of the bridge must match the design and needs to be ready on time because any error can cause a problem for the planning. This is very important because in the whole time of installation of the movable bridge spans the waterway is close to navigation, so any delay can bring many consequences.

For movable bridges, there is a very wide range of components of various fields and some of them quite specific and detailed, so it is common that these are from different manufactures and from different places. Therefore, the fabrication methodology requires a good planning.

After the installation of the operating system and leaf has taken place, the leaf is often set in fully open position. To keep this position, a locking device will be used so that the operating mechanism is not under influence of the wind loads and work can still be carry out until the bridge is taken into normal operation.

A vital aspect to consider (as well in fixed bridges) is the installation sequence. In movable bridges, if the bridge is operated before the installation of the whole components and counterweights or if the bridge is left in opened position for extended period of time, the loading will change and consequently the whole mechanism will work in a different way as the designed.

3.8. MAINTENANCE

The maintenance and inspection is always a key part for a long and safe life of any structure and so in the design this is a fundamental criteria. Nonetheless movable bridges have some particular maintenance issues referring to the interaction of the structural and the electrical and mechanical components, which poses a more significant risk of mal-function of the movable bridge. Having this into consideration, every movable bridge is designed differently and it has unique features, so each one should have its own Operation and Maintenance Manual.

Normally when inspected, movable bridges have some common problems:

- Worn machinery;
- Broken machinery supports;
- Lack of lubrication;
- Misalignments of the locks and bearings;
- Problems with the power of the motors;
- Balance of the span.

With a large common history of movable bridges problems, it is possible to put additional thoughts in these topics in future designs. Nonetheless, it is almost impossible for some of them to find a solution, because the problems are due to the pass of time and the only solution is to know how to approach these with the right knowledge in every field (mechanical, electrical and structural).

One of the most important facts that contributes to reliability issues is the lack of a regular programme of lubrication.

4

CASE STUDY – GREAT YARMOUTH THIRD RIVER CROSSING

4.1. BACKGROUND

Great Yarmouth is a coastal town, situated on the southeast of England. The Great Yarmouth Third River Crossing is recognised by the Council, Norfolk and Suffolk Local Transport Body, New Anglia Strategic Economic Plan[2] and the A47 Alliance as a strategic priority for unlocking future economic growth in the area. This will provide much-needed connections between the Strategic Road Network and the fast-growing areas around, like the Enterprise Zone and Energy Park. It will also ease existing congestion problems, create a direct link into the southern part of the peninsula and improve accessibility in Great Yarmouth, including access to the seafront, South Denes and the outer harbour areas.

The Great Yarmouth Area Transportation Strategy[3] describes a Third River Crossing across the River Yare in order to relieve congestion on both existing bridges and strategic roads. In addition, it will also improve accessibility within the urban area and enhance the non-motorised user facilities in the area.

Fig. 4.1 - Great Yarmouth port (Mike, 2004)

[2] Carried out in 2014, sets out the ambition of the Local Enterprise Partnership (LEP) to deliver more jobs, improved skills, new business and housing.

[3] Carried out in 2009, the Great Yarmouth and Gorleston Area Transportation Strategy examined a wide range of strategic solutions to the areas transport problems and opportunities.

4.2. Site Description and Constraints

Great Yarmouth is part of a larger economic sub-region with a strong economic heritage including manufacturing, food and drink processing, tourism and leisure industries.

Despite its relatively isolated location, confined by the North Sea to the east and the two rivers Yare and Bure to the west, it is an important employment centre and tourist destination, being highlighted as a key growth location within the New Anglia LEP's[4] Strategic Economic Plan. The area has been designated one of six UK Centres for Offshore Renewable Engineering and has two Enterprise Zones selected for energy businesses, offshore engineering, ports and logistics. One is at the Port and the other at Beacon Park. In addition to the port Enterprise Zone area, a Local Development Order has been agreed for the entire of the South Denes wider area, in order to boost employment growth.

There are only two crossings for this peninsula with two single carriageways lifting bridges. The access to the north of the town centre is carried out through traffic on the A12 which crosses the River Yare by Breydon Bridge and through the Haven Bridge is provided access to the peninsula from the south and western part of the town. The outcome is a 4km distance from the main industrial areas to the nearest bridge, both vehicles traffic, pedestrian and cyclists traffic.

The Breydon and Haven bridges currently fulfil a daily traffic of around 70,000 vehicles with about 5,000 vehicles using the bridges in the peak hours. There has been a steady but modest growth in traffic since 2003 when the possibility of a third river crossing was first explored. Currently, with additional development pressures, river crossing traffic is anticipated to rise to between 80,000 and 100,000 vehicles per day by 2030. (Mott McDonald, 2009)

[4] Local Enterprise Partnership

Fig. 4.2 - Great Yarmouth

The River Yare is relatively narrow, between approximately 80m and 100m wide, and mostly straight with some minor bends. Also, there is a series of active quayside berths and hards along the river extension. These accommodate various types of ships, some being of great proportions. As the designated swinging areas have length restrictions, the larger ships must back along the river either on arrival, or departure. In addition, it has to be considered that tidal flows can be strong and strong fresh water flows in the river can increase current speed significantly.

For the proposed scheme, before considering a movable bridge it was considered the possibility of a high level fixed bridge. This option was rapidly putted aside due to some important factors and constrains, listed below:

- The maximum air draught that can be provided within the confines of the area available for the approaches, is limited and this would impact on the size of vessels that could pass beneath.
- It would be necessary rearrangement of existing berths to accommodate the largest vessels.

- Construction and land acquisition costs that would be needed for the fixed bridge structures would be much greater than for movable bridges.

The environmental impact of the fixed bridge option has not been studied in detail at this stage but the constraints presented are enough to make it a reasonable decision. However, the high-level structure needed would cause a significant impact to residents and business.

4.3. NAVIGATION TRAFFIC

The feasibility of the analysis of ship traffic in its relation to bridge clearances and operation needs to be checked for being one of the first step and most important and critical part of the study. For this purpose, further information is needed about every ship's movement and details.

Great Yarmouth Port Authority (GYPA) maintains a database of ship arrivals, movements and departures covering all commercial ship movement and details since January 2004. A copy of this data was received from GYPA with a limited list of vessel air draughts for a representative set of ships using the harbour provided by the GYPA later. With this information, a navigation study by Mott McDonald and a Navigation Simulation Study by HR Wallingford were carried out (2009).

The recording time of the study was commencing in late October 2003 and finishing in early October 2007. The results of the shipping data analysis, estimated the likely requirement for bridge openings through the average number of daily passes that the bridge is expected to get. It also provided a basis for the various air draught, making it possible to set a level of bridge that can maximise the number of small craft passes without the bridge being opened.

It is concluded that an opening of 50m should prove sufficient for commercial vessels traffic, when shipping is moored on both sides of the river.

4.4. DESIGN CONSIDERATIONS

4.4.1. BRIDGE LOCATION

There are two main aims in this project that need to be minimized:

- Impact on existing infrastructure and vehicular traffic flows;
- Impact on ship movements.

For the first aim, due to the existing main road layout and physical constraints placed by surrounding development, the location of the new crossing needs to be located as far downstream as possible, whilst providing a direct connection to the A12 Road network. For the second aim, to provide a safe passage of ships it has to be as far upstream as possible. This causes a conflict between the two main aims, so it is necessary to reach a balance. After some study and discussion, it was identified that the best solution would be at the area shown in Figure 4.3.

Fig. 4.3 - Area of interest and Potential locations

The most economically viable link with the main road network is at the Harfreys Roundabout on the A12, which comparatively with other crossing locations is the one with a relatively small impact on existing residential and industrial infrastructure and fairly direct connections to the existing road network on either side. Within this area, the northern location corresponds to a large drop in ship movements, reducing substantially the number of bridge openings. It was concluded, there, that this location is the best option.

Furthermore, it is preferred that the new structure would not be on a curve of the river because it requires a larger clear span, which would incur a greater cost. It is also noted that highway alterations may be required outside the area illustrated to achieve effective links into the existing network.

4.4.2. Geometry – Alignments and Clearances

After a lot of considerations about the horizontal and vertical alignment of the approach road and the optimization of the location, the recommended option is shown in Figure 4.4, on which the sketches for the various movable bridge options are based.

For the location selected above, and to simplify the design and construction of the movable bridge it was selected an alignment completely normal and straight to the river. This will provide a better navigation of the vessels.

The vertical alignment of the preferred route allows Southtown Road on the west approach to continue unobstructed, enabling a minimum headroom of 5.3m provided above the same.

This alignment which incorporates a vertical curve connecting the approach gradients brings various advantages with it, improving aesthetic appearance, positive drainage and good sight of the boats as approaching the bridge.

Also, with this arrangement the road level at midspan provides an increased air draught of a minimum 7.5m underneath the bridge when in closed position. This would allow small vessels to pass under the bridge without opening the bridge, reducing the number of times it would need opening.

As concluded before in the ship analysis, the need to maintain access for shipping requires a clear width of at least 50m and at least an air draught of 40m above Mean High Water level.

For bascule and swing bridges the air draught is limitless when the bridge is in open position. However, for the vertical lift bridge the air draught is limited and has to be studied.

Afterwards, a comparison between bridge options will be carried out. For this it is considered a clear opening for navigation of 84m, i.e. the full width of the existing channel at the proposed crossing location and the minimum width necessary of 50m.

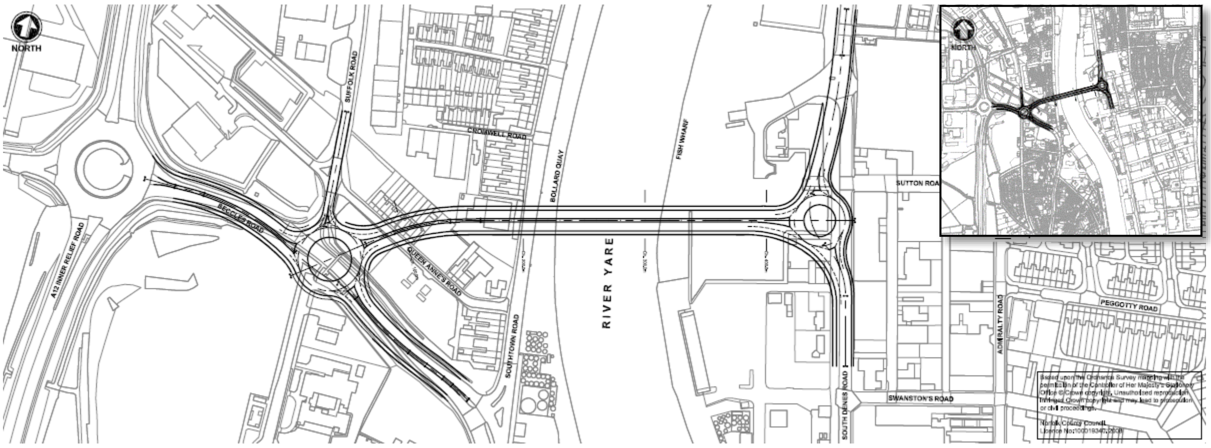


Fig. 4.4 – Alignment of the proposed bridge (Mouchel/WSP, 2017)

4.2.3. Design Service Life

The specified design life for the major components of bridge structures in the United Kingdom and thus in this bridge is 120 years. As this is a movable bridge some specified components like mechanical and electrical components have different design lives and have to be taken into account. It is also noted that these machineries need to have constant maintenance and replacement throughout the life of the structure. The specified design life of each component is shown below:

Table 4.1 - Components design service life

Component	Life (Years)
Piled Foundations	120
Reinforced Concrete Substructure	120
Reinforced Concrete Superstructure	120
Steel Superstructure	120
Bridge Waterproofing	30
Bridge Surface Protection	30
Fenders	15
Fixed Span Bearings	60
Movable Span Bearings	60
Hydraulic Cylinders	60[5]
Electrical mechanisms	60
Hydraulic Motors and Power packs	30
Hoisting Ropes (Lift Bridge)	30

4.5. Movable Bridge Options

There are several types of movable bridges that can be chosen to this proposed route as described in section 2.2. Of all the main types described here, transporter, retractile and pontoon bridge will not be considered in the detailed discussion below.

The retractile type is rarely used today and would not be an economic solution, where the spans would have to be supported above the side spans rather than on land.

The transporter type is also rarely used today and particularly applicable where the span is very long and other methods are considered difficult to construct. This is not the case, though, so this solution was put aside.

Pontoon type bridges are mostly temporary and in this location, it would not be considered feasible.

[5] Cylinders may require re-profiling after 30 years

4.5.1. Option 1 – Bascule Bridge

There is a large list of bascule bridge types that can be seen in detail in Chapter 2. The main features of a bascule bridge can be summarised here:

- Commonly used in Europe and in the UK;
- Economical for small to medium range spans, i.e. 10 – 50m. However larger bascule bridges have been and still being constructed;
- More installed power needed than other types (more wind load);
- Minimum occupation of river frontage;
- No limit on air draught when in open position;
- Superstructure area limited to navigation span only;
- Mechanically simple in single leaf form with no locking system required;
- Bascules with underneath counterweights need a counterweight chamber below deck level and have a minimal visual impact when in a closed position and significant visual impact when raised.

For most of all types, it is usual that the arrangements have counterweights to minimise the power required for the lift operation of the bridge. As this bridge is a large moving structure, it is considered that a counterweight design would be the most economical and best choice.

Strauss bascule bridges have very important disadvantages as these are mechanically more complicated, difficulty maintenance and the overall appearance is declared as a 19^{th} century artefact. Such a bridge would not be suitable for a structure in the 21^{st} century and in this development area.

Scherzer bascule bridges varies from his counterweight design that can be overhead and underneath. This type is still in common use today and can be found all over the UK. Those with overhead counterweight are not very appealing in aesthetics and in this case as the client wishes, the bridge has to have an attractive appearance. Furthermore the roller tracks and the circular girders which roll on them, are susceptible to fatigue, and the maintenance repairs of these materials are extensive besides if not chosen correctly problems may occur. So it is concluded that a Scherzer bascule bridge is not suitable for this crossing.

An alternative type would be a Dutch bascule type, but these have a significant visual impact and as already explained, the client does not affection this idea. Nevertheless, this can be used as an opportunity where an iconic bridge is thought.

The bridge types examined are therefore limited to:

- Trunnion bascule bridges with underslung counterweight
- Trunnion bascule bridges with overhead counterweight

It has to be noticed that when the deck is close to the water level, as in this case, a counterweight pit is necessary which will be below water and therefore more expensive. However, if an overhead counterweight is used the refereed pit is not required and the superstructure and substructure (apart from the foundation) is kept above the river water level.

4.5.1.1. Leaf Types

Bascule bridges can be of single or double leaf. Single leaves are mechanically simpler to construct than double leaves. However, the foundations required would be very much deeper to support the counterweight pier (if underneath counterweight) when the bridge is in opened position and if the spans are relatively close to the water, the pit housing the counterweight would have to be founded many meters below river bed level, which would be expensive to construct. Moreover comparing with the single leaf structure, the construction of two leaves would provide two small counterweights and consequently a reduced depth of the pit.

Even though the double leaf has the disadvantage of requiring two sets of operating equipment (including as well the two small piers) and nose locks for conjoining, the costs incurred for this will not be much different than the nose pier that would be necessary to construct a single leaf type bridge in this location.

An important matter to be considered is that the wind resistance for the machinery to overcome in the opening of the bridge is much greater for the single type than for the double. Furthermore, the size required for a single leaf structure in this location would be out of scale with the surrounding infrastructures.

With this it is concluded that in this location a twin leaf bascule is more suitable than a single leaf bascule.

4.5.1.2. Structure Type VS Opening Span

With the construction of a twin leaf bascule bridge with a clear span of 84m (full width of the river in this location) a pit counterweight would have to be constructed many meters below the bed level since this is a low level bridge. Aside from this being very expensive, the foundation piles might prove impossible with conventional cofferdam construction and it would be needed to use caissons. Moreover, the opening for the twin leaf bascule has a shorter overall cycle time between the opening and closing operation of the bridge. This method will bring large costs and risks so it is concluded that a bascule bridge with this span clearance would not be the best solution.

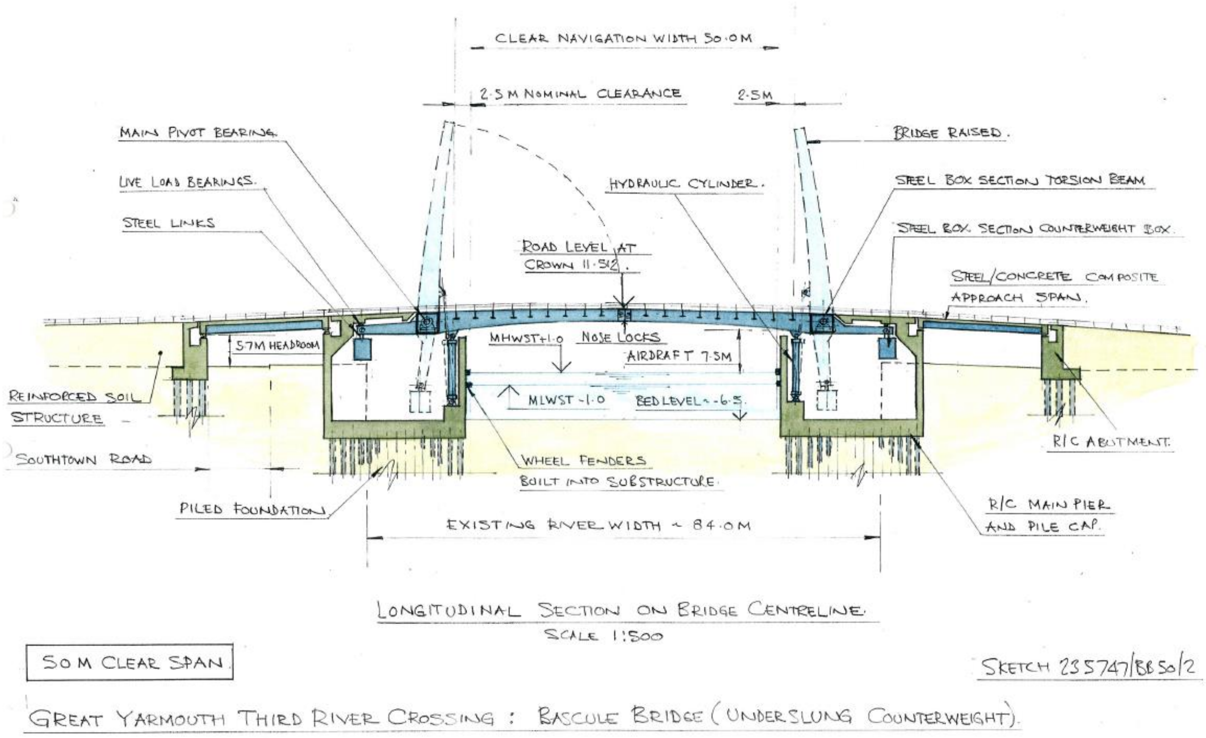


Fig. 4.5 - Bascule bridge option – 50m clear span (Mott McDonald, 2009)

4.5.2. Option 2 – Swing Bridge

The main features of a Swing Bridge can be summarized as:

- Commonly used in Europe and in the UK;
- Economical for all spans, i.e. 10 – 300m;
- Less installed power needed than other types (less wind load);
- In open position occupies a long length of river frontage;
- Collision protection is needed along the full length of the superstructure - a serious disadvantage in this location where ship impact loads are potentially very large;
- No limit on air draught when in open position;
- Tail or back span typically 30 – 40% of the navigation or main span so a longer superstructure needed than other types;
- Superstructure and substructure (apart from foundation piles) can be kept above the River water level which facilitates construction - although where ship collision loads govern foundation design, this is not always possible;
- Where the opening span is sited on a pier in the navigation, it is inaccessible for maintenance except by boat when in the river open position;
- Bridge needs a wedging system – mechanically more complicated than other types, and thus potentially more labour intensive in maintenance;
- Minimum visual impact.

4.5.2.1. Leaf Types

Swing bridges can also be single or double leaf. The double leaf type brings more complexity to the design as they require mechanical devices to provide rigidity under traffic loading. Nevertheless, there are many cases of swing bridges with this form and function without any trouble.

For the case of a twin leaf arrangement, would not bring benefits because for this level of clearance a simple single leaf is adequate and does not require unnecessarily complicated mechanics. Also, when in open position it would only occupied one of the banks instead of both.

4.5.2.2. Structure Type VS Opening Span

For a swing bridge with a clear span of 84m, the total length of the structure, 200m, would require that the main pivot would be at the quayside and an enormous space along the quayside would be necessary to keep the main and back span when in open position. Hence the demolition of residential parcels would be mandatory if the main pivot rotates on the west side and commercial parcels if it rotates on the east side. The costs for the parcels acquisition and demolition will be much more expensive than for a swing bridge with a clear span of 50m.

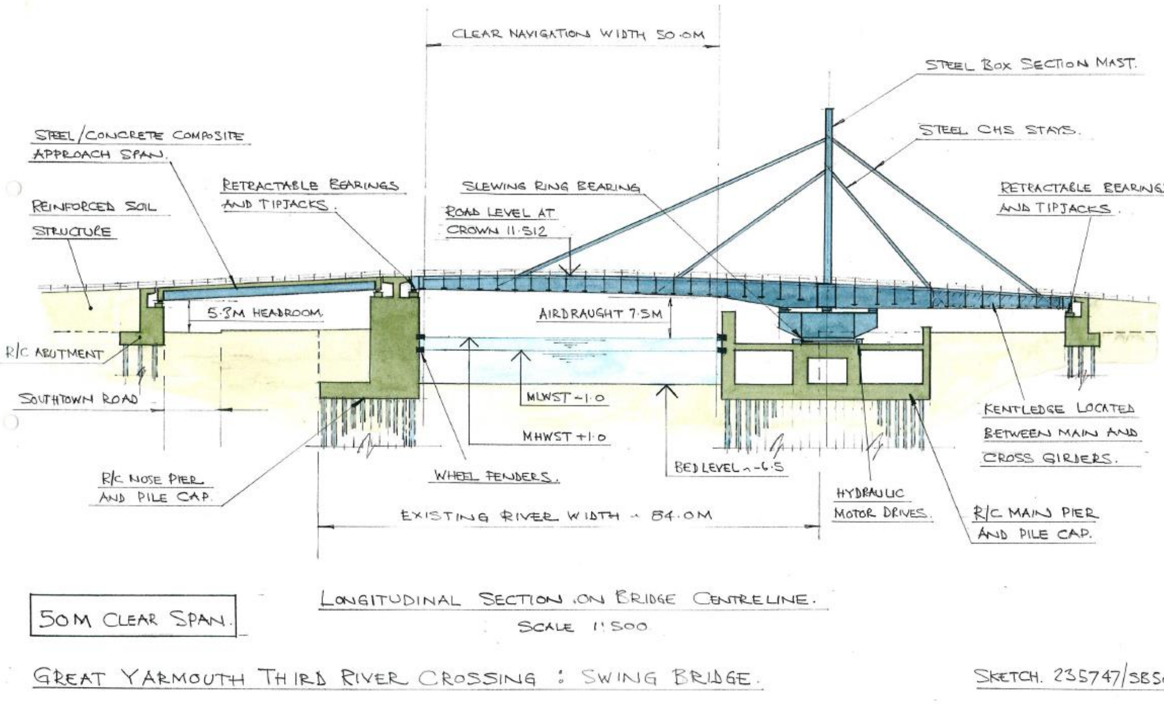


Fig. 4.6 - Swing bridge option – 50m clear span (Mott McDonald, 2009)

4.5.3. Option 3 – Vertical Lift Bridge

The main features of a Vertical Lift Bridge can be summarized as:

- Less used in Europe and in the UK. Commonly used in USA.
- Normally used for larger clearances.
- Economical for medium to larger spans, i.e 30 - 150m.
- Installed power needed similar to a swing bridge.
- Minimum occupation of river frontage.
- Air draught not unlimited. Because the vessels using the river are commercial craft, including tall masted sailing craft, the lift height would need to be large and the lift towers tall as a result relatively expensive.
- Superstructure area limited to navigation span only.
- Superstructure and substructure (apart from foundation) can be kept above water level, facilitating construction.
- Mechanically simple but the rope systems used in the lift towers needs regular replacement.
- Significant visual impact (because of the lift towers).

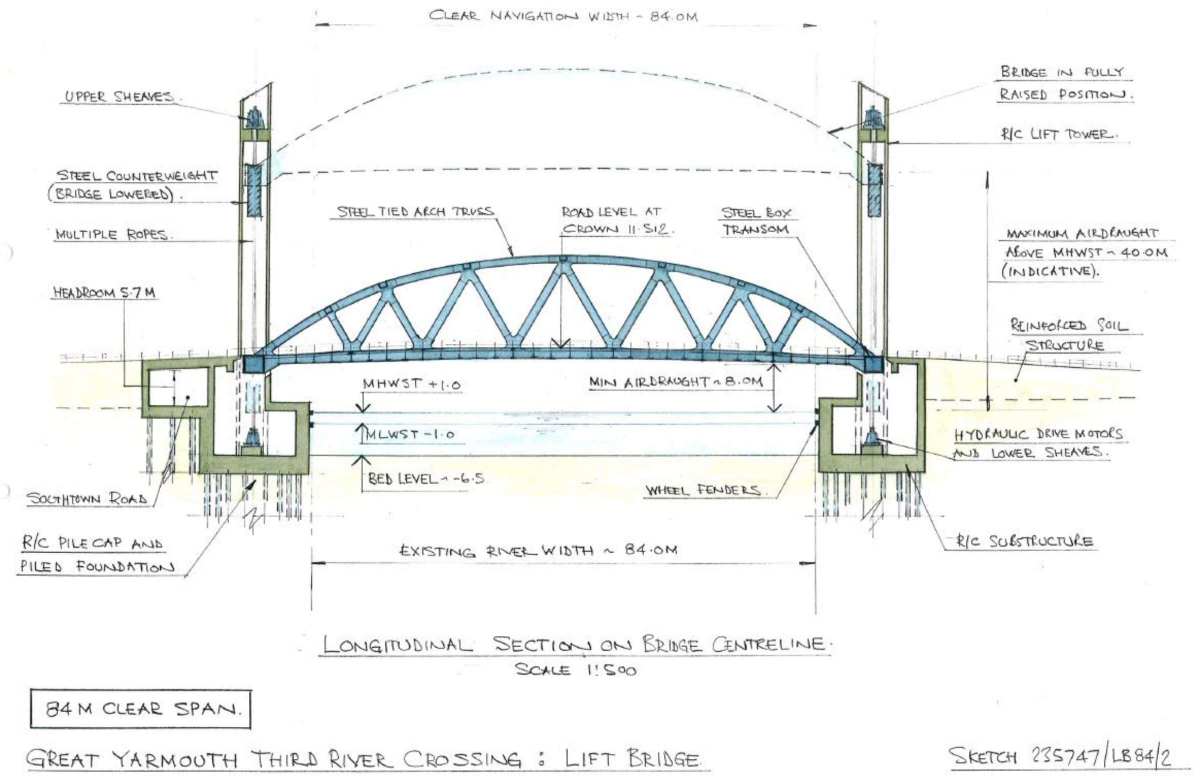


Fig. 4.7 - Vertical lift bridge option – 84m clear span (Mott McDonald, 2009)

4.6. LOADS

The loading adopted for bridge design and accordantly in this paper is that specified in Eurocode 1: Actions on structures, which for bridges is quantified in part 2 (EN 1991 Part 2) and UK National Annex's to Eurocode. The actual and applied loads to be used should be thought further in detailed design proceed for the final option and calculations. Nevertheless, for a movable bridge it should be taken into account the loads that are not stated in the Eurocode.

4.6.1. PERMANENT LOADS

Self-weight is the basis of any design and in the case of movable bridges it has influence in the superstructure and in the operating mechanism. This is one of the loads that defines the required torque of the engine. This load is taken into account for the standard operation load of the operating mechanisms, according to the form of the superstructure.

4.6.2. WIND

Perhaps the most important load of all, wind causes different behaviour on the bridge when this is in open and closed position. This impacts the installed power to operate the bridge and as a result the capital cost. Particularly for bascule bridges that, when raised, the entire deck area is subjected to the wind loads. So, the installed power for the hydraulic or mechanical system has to be able to overcome the wind loads on the raised deck/s. Swing and vertical lift bridges are not so sensitive to this.

It was informally discussed with the Harbour Authorities the changes in weather conditions which changes the movements of the commercial ships and consequently the raising of the bridge. For this section, it was recommended wind speeds and times for the operation of the bridge. These have to represent a reasonable balance between the installed power of the mechanism and the availability of bridge operation therefore was selected 45mph or 72km/hr – i.e. with gust speeds of up to 70mph or 112km/hr. This in the Beaufort scale refers to a force of 8 between 0 and 12 which is descripted as Gale (Royal Meteorological Society, 2015) with moderately high waves of greater length, edges of crests break into spindrift and foam is blown in well-marked streaks along the direction of the wind.

For checking the stability for wind loads it has to adjust the basic hourly mean wind speed accordantly with geographic location, topography, height, etc. For Great Yarmouth is used the assumed basic wind velocity value of 23m/sec (approximately 83km/hr) accordantly with the National Annex to Eurocode 1 – Action on Structures.

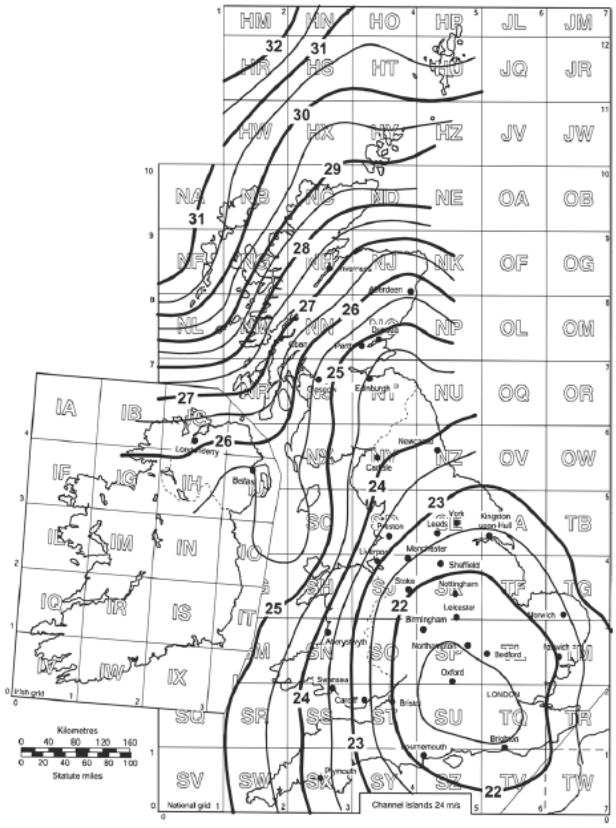


Fig. 4.8 - Value of fundamental basic wind velocity before the altitude correction is applied (NA. 1)

In this case it must be considered the situation where the bridge spans are raised and stay opened for a large period for maintenance purposes, as for example, repair or replacement lifting mechanism components. It is imperative that the bridge structure can secure the raised spans and be stable to the wind loads defined above for the whole process.

4.6.3. Ship Impact Loads

The possible ship impact that can occur in this part of the river will generate very large forces that would be severe to the design of the substructure. It is recommended that the movable spans are checked in the opened and closed position for a collision load from a small vessel as these may not sustain the impact forces. Also, if applicable it is recommended to check for sided fixed spans as well.

Bridge design utilising these forces should enable the bridge substructure to survive the worst credible collision scenario.

A protection in the side spans can be used with a fendering system, to prevent and minimize the effects of vessels colliding with them.

4.6.4. Snow

Snow load with live load is not a significant combination for bridges in this location, but for movable bridges the combination of snow load and dead load can impose significant complications. It is designed according to the Eurocode for the closed position.

4.6.5. Traffic Load

Traffic load is one of the governing loads for the design of the superstructure and is designed according the Eurocode for the closed position. For the operating mechanism this load can be neglected as there is no traffic present on the bridge during operation.

4.6.6. Temperature

The variation of temperatures affects directly the materials of a structure and it can cause displacements or stresses at the elements. Temperature is related with the geomorphology of each area. The temperature effect is not that influential at this stage so it was not considered here. However, in a later stage this has to be considered.

4.6.7. Hydrodynamic Loads

A hydrodynamic analysis has to be carried out in the river piers as these are subjected to river currents and tidal flows. Scour from tidal flow is a long term effect and can become most critical than the ship impact loads discussed above, so a detailed design as to incorporate this effects.

4.6.8. Dynamic Loads

The detailed design will have to consider the performance and dynamics loads of the bridge, due to the acceleration and deceleration of the operating mechanism. There are requirements for these loads in normal situation and in emergency case. The load introduced by this has no influence however on the force distribution between the two mechanisms.

The location indicates a low seismic zone which has a low probability of seismic concerns in the current design, apart perhaps some specific components, like machinery which behaviour is not ductile (that are considered within the electrical and mechanical engineering). Thus the seismic design is beyond the scope of this work.

In a later phase, in detailed design, when carried out a dynamic analysis it should be considered all positions of the bridge (opened and closed).

4.7. Aerodynamic Considerations

The aerodynamic performance of movable bridges is an important matter since the dynamic response of the structure and hydraulic system under wind loadings are of primary importance to successful operational performance. In the current work it was not carried out any aerodynamic evaluation, but it should be in further development of the bridge design. For this, some studies within the evaluation of the effects of vortex shedding have to be complete, buffeting and flutter for the proposed deck with a wind tunnel testing.

4.8. Superstructure Types

4.8.1. Structural Forms

Current movable bridges commonly use steel orthotropic deck construction as it is possible the most suitable option for the purposes of these type of bridges that are subjected to regular changes of stresses during their operation. Such decks comprises a steel deck plate stiffened on the underside by longitudinal and transverse stiffeners. These have the disadvantage of being far more expensive to fabricate than open grid decks but in the other hand they are lighter and cheap to maintain and paint. In addition they have a clean external appearance.

For bascule bridges, the form of this steel orthotropic deck normally is of the deck span type, i.e. the main longitudinal members are underneath the deck. For this members it is possible to have various arrangements, like I section plate or box girder form and can be integral with the supported deck. Deck span types though have much more attractive appearance, requires more construction deck depth. Since in this case the additional depth is small, it does not present a major problem to the air draught require for the passage of small vessels.

For swing bridges it is possible also an arrangement with a cable stayed main span. This is an economical solution because the stays supports the main and back spans. A protection of the masts and stays have to be carried out for vehicle impact.

For lift bridges the solution adopted varies normally between trusses or tied arch main members. This because these members are lighter and more economical than the ones used for bascule bridges. As the span is vertically lifted the main objective is to minimise the constructive depth of the deck so the most common arrangement is a ‘through deck’, i.e. the main longitudinal members are above the deck. This form needs a carefully detailing for an acceptable appearance and protection of these members from vehicle impact.

4.8.2. Materials

Major structural components use steel with the purpose of minimise the weight. The surfacing used in moving spans has to be such that minimises the weight. For swing and vertical lift bridges, it is normally used a thin layer of 30mm of mastic asphalt overlaying a waterproof membrane if necessary. This mastic asphalt assists in load distribution which reduces fatigue stresses in the steel deck plate beneath. Another option for the surfacing is epoxy-bauxite (aluminium oxide), that can serve both purposes, waterproofing and surfacing and it’s laid directly onto the blast-cleaned steel deck plate.

4.9. Substructure Types

4.9.1. Structural Forms

The foundations location will be different according with the length of the clear span.

When the entire navigation channel is clear opening span the substructure can be located in both existing banks of the river. For vertical lift, swing and bascule bridges with overhead counterweight the common substructure is above the water level which facilitates the construction process significantly. For a bascule bridge with underneath counterweight it is necessary a pit to house the counterweight when in open position. Due to the huge size of the span and counterweight that the bascule with this clear span would require the pits (main piers) to be much more below than the river water level, in cofferdam.

For swing and bascule bridges with overhead counterweight the common form of structural substructure would be a reinforced concrete slab above water level, supported on a raft of piles which can be driven or bored into the river bed. For bascules with underneath counterweights a reinforced concrete pit is founded at bed level, on spread footings or piles.

For this bridge the possible crossings, two possible foundation locations are considered as referred before, 50m and 84m (full width).

4.9.2. Materials

The material used for the substructure is as noted before reinforced concrete. Concrete is used due to his extremely capability of suffering high compressive stress before any type of cracking or failure occurrence which makes it ideal for supporting large masses. Also, the relation between cost and its efficiently is much more low than for other material.

Since the majority of the foundations are sited in a marine environment, the concrete quality and reinforcement cover needs to be chosen accordantly, i.e the value for cracks width at Serviceability Limit State as to be limited for marine environments (exposure classes according to EN 1992-1-1).

4.10. Foundation Types

A preliminary engineering assessment for the various options of crossings was carried out by Mott Macdonald based on the ground investigation information done previously by Norfolk County Laboratory. In this section is carried out a re-assessment to this conditions and comprised the ground risk and possible foundation solutions for the preferred options before chosen.

The route of the proposed crossing runs from the intersection between Queen Annes Road and Suffolk Street, crossing Southtown Road on the Western Bank of the River Yare. The bridge is proposed to cross over to the eastern bank of the river at Fish Wharf with an approach road over South Denes road. The geological units expected to be encountered across both sides of the river are:

- Alluvium
- Blown Sand
- Breydon Formation
- North Denes Formation
- Corton Formation
- Crag

The geotechnical information is shown below connecting the strata with the depth of each geological unit:

Table 4.2: Summary of ground model (Mott McDonald, 2009)

Strata	**Depth (m bgl) of top of stratum**	**Depth (m bgl) of base of stratum**	**Boreholes used to determine ground conditions**
West Side			
Made Ground	0.0	1.4	TG50NW/28 TG50NW/29
Breydon Sand	1.4	3.0	BER31_05 BER31_10
Breydon Clay (organic)	3.0	4.0	BH 101
Corton Formation	4.0	11.5	BH 102 BH104
Crag	11.5	25.0	NCC301 TP 101 TG50_3
River Section (inferred from further up river)			
Alluvium	0.0 (River bed)	1.0	PH2_BER14_BHS2
Breydon	1.0	6.0	PH2_BER14_CPT01 PH2_BER14_CPT02 PH2_BER14_CPT03
Corton Formation	6.0	16.0	PH2_BER14_BH04 PH2_BER14_BH05
Crag	16.0	24.0	PH2_BER14_BH06
East Side			
Breydon Formation	0.0	7.5	BER12A_08, BER12A_09 BER12_11
North Denes	7.5	12.1	BER12_12 BH 103
Crag	12.1	15.0	WS 107

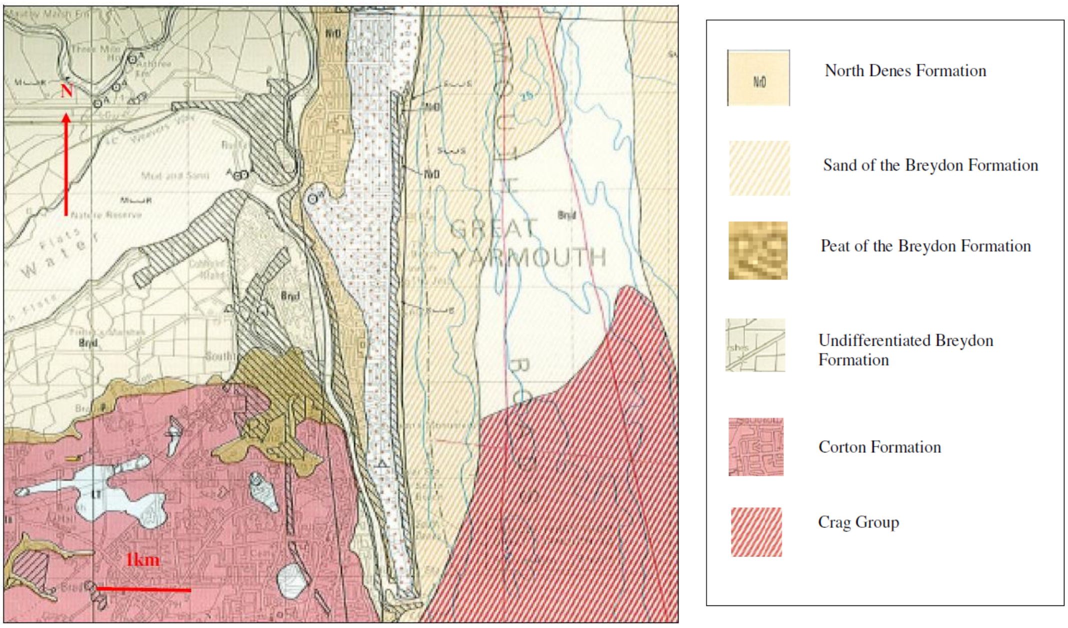


Fig. 4.9 - Geological Map of Great Yarmouth District (British Geological Survey,1990)

4.10.1 GEOTECHNICAL CONSIDERATIONS – 84M CLEAR SPAN

- **Shallow Foundations Options**

Both sides of the River Yare comprise top soils of Breydon Clay Formation which consist of potentially highly compressible soils of turf, clay and silts which depth are recorded at 4m below ground level on the western side and 7.5m below ground level on the eastern side.

These soils are problematic because they have poor consolidation properties. They suffer from high and variable settlements under load unless ground improvement is carried out. Peat in particular is partly decomposed and fragment remain of plants that have accumulated water so this has a high, low shear strength to loads and a high coefficient of secondary compression which means settlement may continue even after primary consolidation has finished and all pore water pressure has dissipated.

When sites contain deep soft soil deposits this brings a major problem because it is likely to affect the foundations and consequently the structure itself. Differential settlement between parts of the structures may occur as a result of variations in strata and material. Furthermore, dewatering of these soils could have a settlement impact on adjacent properties.

Established the previously observations and comments it is concluded that shallow foundations are considered highly likely to be unsuitable for the proposed structures. The vertical and horizontal loads would be much higher than the provided resistance of the shallow foundations and therefore a suitable deep founding will be needed.

- **Deep Foundation Options**

Deep foundation options includes a deep raft or a piled solution.

Based on the information obtained by the investigations, deep foundations are likely to be suitable founding strata with Corton Formation and underlying Crag on the western side and river section and North Denes Formation and underlying Crag on the eastern side.

The Corton and North Denes Formations are variable deposits with sand, silt and clay soils. The uncertainty of the deposits locations and the large and variable loading of the bridge could cause large differential settlements issues for raft foundation options. Therefore if this solution is to be considered a detailed ground study has to be carried out. There is however a significant risk that this would not be feasible for a bridge.

A piled solution would penetrate through the Corton and North Denes Formations into the underlying Crag which is recorded at 11.5m below ground level on the western side and 12.1m on the eastern side. Bored cast in situ concrete or driven piles are both likely to be suitable to transfer the loads from the bridge to the competent founding strata of adequate bearing capacity for this case.

- **Excavations**

The excavations needed for the main pier chambers will have to be very deep due to the large side of machinery and if case the counterweights. So the base would lie approximately around the level of the Crag Formation. To get to the Crag in the west side of the river it will be needed to cut through Breydon and Corton formation.

4.10.2 Geotechnical Considerations – 50m Clear Span

- **Shallow Foundations Options**

In the river section the top soils beneath the river bed, Alluvium and Breydon Formation, consist of potentially highly compressible soils of turf, clay and silts which depths are recorded at 6m below. Is noted that it exists the possibility of deeper deposits locally.

As described as well in the considerations of the shallow foundations for the 84m clear span, deep foundations are the suitable choice for these type of soils.

- **Deep Foundation Options**

Based on the information obtained by the investigations, deep foundations are likely to be suitable founding strata with Corton Formation and underlying Crag at 16m below river bed at the river section.

As in deep foundation options for the 84m clear span in section 4.4.8.1, the best reliable choice is the piled solution as well.

- **Excavations**

The excavations needed for the main piers chambers will be inserted with the base at approximately at the level of the Corton Formation. Piles will be needed to go through the Alluvium, Breydon and Corton Formations and be installed in the Crag.

4.11. Ship Collision

For the options where the opening span piers are in the navigation channel there is a high probability of vessel collision. Therefore, the substructure type of reinforced concrete slab constructed above water level is unsuitable. So, the only alternative is the reinforced concrete pier with the base slab at or below bed level, regarding that this is merely for swing and bascule bridges.

For the lift bridge option, the substructure is sited behind the quay lines so it is thought that the occurrence of a vessel collision would cause two situations. One is the collision with the quay itself, that would cause a major local damage but probably not put the bridge out of order and the second is if the same collision occur instead of only on the quayside but in the bridge as well. This could put the bridge out of action for a long period of time. Nevertheless, this situation is much more unlikely to happen.

Because of the potential consequences of a ship collision, the piers/structure must be provided with a fender system to ensure the minimal damage possible to the bridge. Their function would be to absorb the energy impact of the vessel and to prevent scratching and scraping and also help with the guidance of the vessel through the correct approaching path. These fenders can either be mounted on independent steel fender piles (steel provides high strength against collision and light in weight) or in the face of the pier depending on the location and the level of protection afforded to the bridge abutments and the constrains that the fenders would place on the operation of the port when constructed. This will be detailed for the selection of the final option.

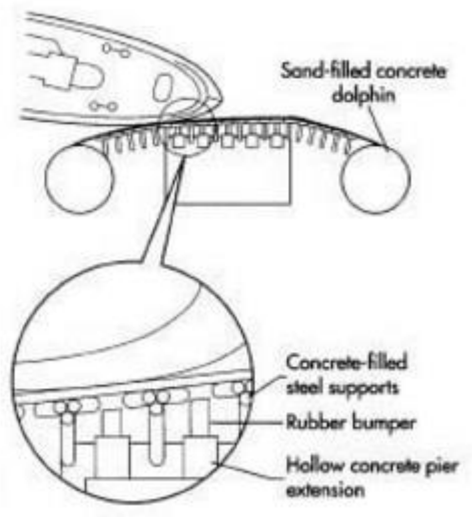


Fig. 4.10 – Example of fendering system (Hall, Unknown)

4.12. Other Considerations

4.12.1. Aesthetics

The various types of movable bridge bring different visual impacts, ones more thoughtful than others. Throughout the study of the different proposals and further design, aesthetics is always present and considered when decisions are being made about the structure and its settings. The most significant topics are described below:

- Bascule Bridge

Bascule bridges types with overhead counterweight have some visual impact being that some types of overhead counterweights have more unpleasant design than others. In these cases, it is taken as an opportunity to design an 'Iconic Bridge'.

Bascule bridge types with underneath counterweight differ when in open or closed position. In opened position only the leaves can be noticed which does not cause a strong visual impact and when in closed position as the machinery and counterweights are hidden so for the casual observer it may seem like a normal fixed bridge.

- Swing Bridge

Swing bridges have the least visual impact of all types if not truss deck. Cable stayed structures are often used in this type of bridges for having an attractive appearance.

- Vertical Lift Bridge

This type of bridges is possibly the worst concerning the aesthetics. These have always large towers to support the lifting span which can be truss type or solid concrete type. The truss type towers tend to have an appearance of the 19th century and the solid concrete type towers are extremely oversized and unpleasant. This can be minimised with a detailed design and if the location of the bridge is in an open landscape.

One aspect that can be of an unattractive particularly is that the truss or tied arch option for the main longitudinal members as is the most economical choice to the span.

4.12.2. Bridge Openings and Traffic Control

A matter to be considered is the time needed for the total operation of the movable bridge and accordingly traffic management. This varies depending on the power rating of the selected lift mechanism.

Generally, movable bridges with spans between 50-85m need a time of approximately 2 minutes to complete the operation of opening and 2 minutes to closing the bridge to navigation traffic. However, the overall time as to be considered the closure time for vehicular traffic and the ship manoeuvre. When the ships enter the river, they must continue their path through the bridge opening to their prescribed berth without interruptions. If for any reason the bridge is not open, for example due to malfunction there would be a necessity for an emergency stop, so it is required an emergency layby berth downstream of the bridge. Furthermore, it is required by the harbour authorities a time interval of at least 10 minutes in advance of the bridge transit to avoid last minute manoeuvres and to give time for the vessels to make decisions in case of some problem. The location of the layby berth in relation to the bridge will be one of the factors that will determine the length of time in advance of the bridge transit the bridge needs to open.

With this, the overall closure time for road traffic in this location it is likely to be approximately 20 minutes.

The 20 minutes that the bridge would have to be close for road traffic will lead to a localised congestion on adjacent roads unless special measures are taken. It is likely that active traffic management will be necessary to divert road traffic over to Haven Bridge for the duration of road closures of the new third crossing.

The layout of pedestrian and traffic controls is one of the key factors to control public and prevent access to potential hazards that can occur in every operation of the bridge. This varies from the type and site of the bridge and the magnitude of pedestrian and vehicle traffic. The distance and type of equipment will need to be in accordance with highway design regulations though the owner may impose additional requirements.

For navigation traffic, it is required navigation lighting and navigational marks on the bridge and, if applicable at the control rooms. These varies depending on the type of movable bridge and have to take into consideration the positioning, arrangement, colours and flashing sequence of these.

4.12.3. Operating Mechanism

The arrangement of the lift mechanism is relatively easy to design for swing and vertical lift bridges. For bascule bridges, this is a little more complicated as the geometry as to proportionate the torque needed to lift the bridge that remains sensibly constant (disregarding the effect of the wind). Two types were discussed, hydraulic cylinders and a rack and pinion drive arrangement using an epicyclical gearbox.

Although hydraulic cylinders is a particularly common solution and the majority of modern bascule bridges are powered by it, is very important to understand the kinematics and motion of the bridge leaves and the change in cylinder loadings. Furthermore, it is necessary to be able to control the speed of the bridge leaves as the bridge achieves high angles of lift. A disadvantage of the hydraulic system is that it is needed two complete sets of hydraulic power packs, one in each bridge leaf, requiring each one of them, maintenance and standby pumps.

The alternative drive mechanism could be curved racks and pinion gears that can be driven from electric motors via epicyclic gearboxes. Nonetheless the electrical systems require regular inspections.

On balance the hydraulic option is considered simpler to design and has been considered further.

4.12.4. Buildability

4.12.4.1. Substructure

For the bascule bridge, the main piers of the bascule bridge require deep substructure and foundations below water level, and besides this in the case of 50m clear span the construction would be sited in the river. Due to the groundwater levels, the excavations works are highly likely to be unstable, so the installation of a coffer dam is probably the most appropriate method of construction to this problem.

For the swing bridge, if the clear span is equal to the river width, there is no need for any works in the river as the substructure is sited in the quays. Deep excavations and foundations are not required as well, so the construction process would not be of much concern. In the case of 50m clear span it would be a little more challenging but as the substructure is not deep it would not be so much as in the bascule type.

For the vertical lift bridge, as for the swing bridge, the construction of the foundations would not be problematic.

4.12.4.2. Superstructure

The primary restraint on the construction of the moving spans is the necessity to limit any works producing operations on the river.

For swing bridges this does not impose any problem because the whole superstructure is constructed in the open position (sited on the quay) and when it is finished it rotates to the closed position, removing significant health and safety risks with erecting and assembling of the structure. For maintenance purposes this is the best type, because it does not interfere with the navigation channel and allows a better safety of the workers. Hence it is possible to save money on the construction of the type of deck if maintenance is easier.

For both bascule and vertical bridges this is not as easy because the fabrication of the deck has to be carried out at the side and then with the help of booms (supported by masts) erected to the right position. For double leaf bascules, as it is considered in this case, erecting heavy steelwork segments over water, means usually additional temporary support arrangements until the final configuration. This imposes a higher level of difficult and risks. As the maintenance of these bridges is harder to carry out commonly, the deck comprises stainless steel to reduce maintenance works. However, this aspect increases capital costs.

4.12.5. Costs

A robust appraisal of cost was carried out by Mott McDonald, based on previous experience of projects of similar types of bridges. This cost only provides an estimation of the capital construction cost of the bridge as this is a very early stage of the project and excludes any works relating to the layby berth, roadworks, connections to the existing network and traffic management measures necessary.

It is also noted that this cost estimation excludes any Optimist Bias or inflation for construction in future years. The estimations were done for the two recommended bridge options:

Table 4.3 - Appraisal costs

Option of Bridge	Cost (£ million)
Bascule bridge	£40m
Vertical Lift bridge	£60m
Swing bridge	£60m

4.12.6. ‘Iconic’ Bridges

When it is necessary, the construction of a new bridge is also an opportunity to innovate and change the design of the typical kind of bridge structure. This type of bridges are labelled here as ‘iconic’ merely for the purpose of distinguish them from the traditional types described before. This is purely up to every person to think if they are iconic or not. In recent years the number of ‘iconic’ bridges designed and built has been increasing, being, however the majority footbridges and bridges located where the river traffic is little and thus a smaller amount of number of openings is required.

Even knowing that the owner preference was for a minimal visual impact, two options were proposed within the owner budget and that would not disrupt the normal activity of this location. These are presented in Figure 4.11 and 4.12.

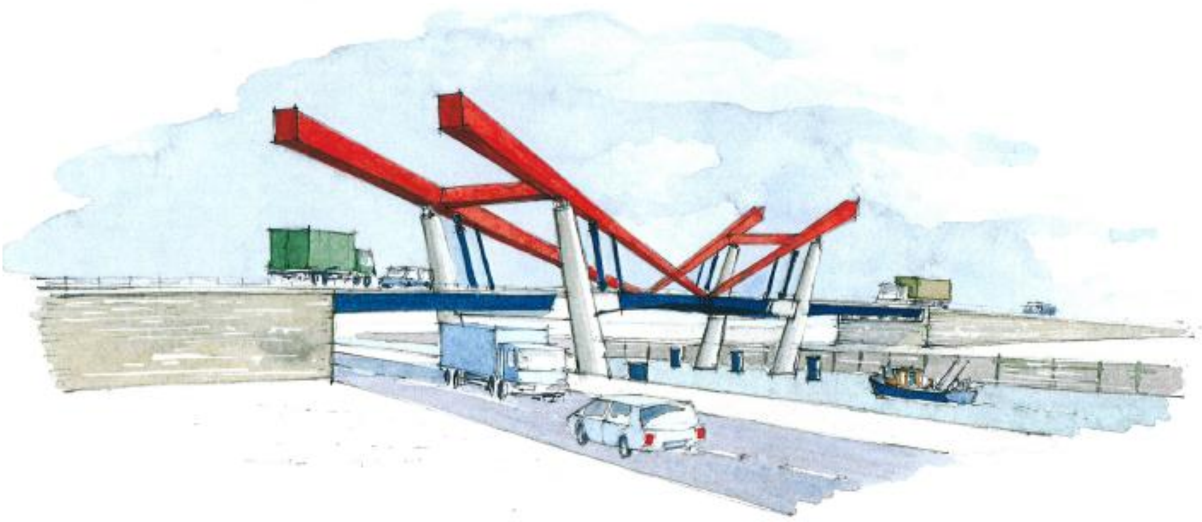


Fig. 4.11 – Bascule ‘Iconic’ Bridge (Mott McDonald, 2009)

Fig. 4.12 – Vertical Lift ‘Iconic’ Bridge (Mott McDonald, 2009)

4.13. Selection – Recommended Option

As a requirement by the project possessor to keep this crossing to a minimum visual impact, the possibility of any iconic bridge and any bridge with big and high elements above deck level were excluded of the selections. Then the only options for the GYTC are bascule and swing bridges.

Even though the swing bridge offers the advantage of not having to deal with the problem of large area exposed leaves to wind loads during survival phase, this option brings several disadvantages, which may not serve this scheme. To achieve the full width of the navigation channel, the length of the bridge deck required would be acquired with a two leaf swing bridge, whose length would be greater than the one for the bascule bridge type. If it is not considered the full width of the navigation channel it would be necessary a system of fendering dolphins to protect the bridge from ship impact. In addition, this option needs more M&E machinery to ensure a correct unlocking of the leaf from its end before starting the rotation operation. Hence, a bascule type bridge will be considered.

After the previous work about the considerations of different solutions, the type of location, vessel sizes and the previous requirement regarding visual impact, the best possible choice is a twin leaf trunnion bascule bridge with underneath counterweight and a 50m clear span over the navigable channel by a fendering system.

4.14. Bascule Bridge

4.14.1. Description of the Design

The design choice corresponds to a counterbalanced double leaf bascule bridge with a clear span of 50m. When each leaf, rotating about fixed trunnions, reaches the maximum opening position through an angle of approximately 83°, the tips are positioned to provide at least 2.5m behind the face of the knuckle wall and an unlimited air draft over the navigational channel of 50m, see Figure 4.13.

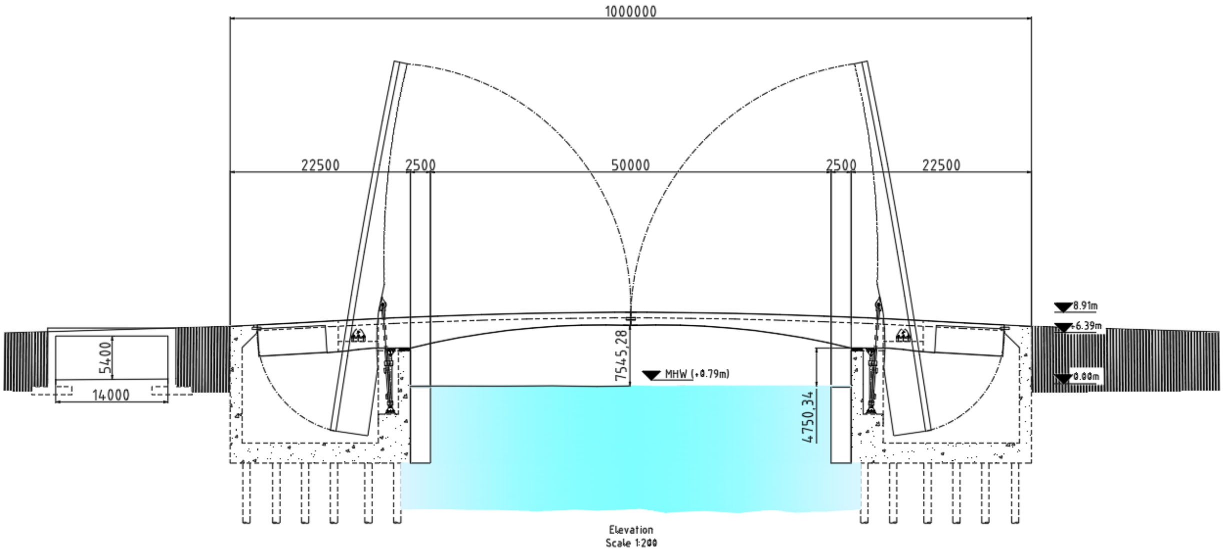


Fig. 4.13 - Elevation of the proposed bascule bridge (adapted Mouchel's report)

4.14.1.1. Highway Layout

The layout had to be considered taken into consideration the anticipated high traffic flow and project owner requirements. For this, three different potential highway layouts were considered:

- 4 lane arrangement with a central reservation;
- 3 lane arrangement with a 'tidal flow' in which the central lane direction is controlled to suit demand;
- 2 lane arrangement.

The preferred option is the 4-lane arrangement, being the best for the traffic flow and for calculations purposes since it is the worst case scenario and if there is any change to a smaller deck at a later stage it will be easy to make the incremental reduction in width.

The speed limit on roads with street lighting (which is taken to indicate a built-up area) is of 30mph. Although there is no requirement for a central reservation for this speed limit, the 1,8m VRS will be maintained along the length of the bridge to facilitate the junction arrangements at either side.

The overall length of the bridge leaves between the heels of the bridge is approximately 100m and with a 68m between the pivot points of the leaves. The overall deck width is 23.4m which carries:

- 2 x 1.5m footways
- 1 x 3 m cycleway
- 2 x 7.3m carriageways (carriageways with 2 lanes each)
- 1 x 1.8m central reservation

- 2 x 0.5 parapets supports

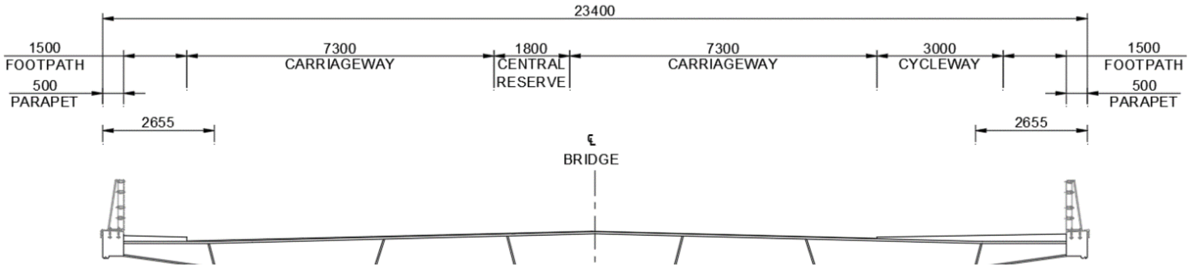


Fig. 4.14 - Layout of the proposed bascule bridge (adapted Mouchel/WSP report)

This layout results in a dead weight of approximately 1030 tonnes per leaf. (1 tonne=1000kg)

4.14.1.2. Form of Deck

Numerous of deck types were considered to check the most feasible arrangement for the present work and are listed below:

- Multiple cell box girders
- Multiple I beams girders
- Multiple trapezoidal box girders

As described earlier, for purposes of a better operating lifting system a lighter deck is needed, so for all the options of arrangements considered use a superstructure comprising a steel orthotropic deck, surfaced with epoxy-bauxite or similar. This surfacing was chosen for the reason that is lightweight, durable and it provides a smaller surface width. The bauxite protects the deck and gives it a higher coefficient of friction reducing potential skids.

1) 4 No. box girder

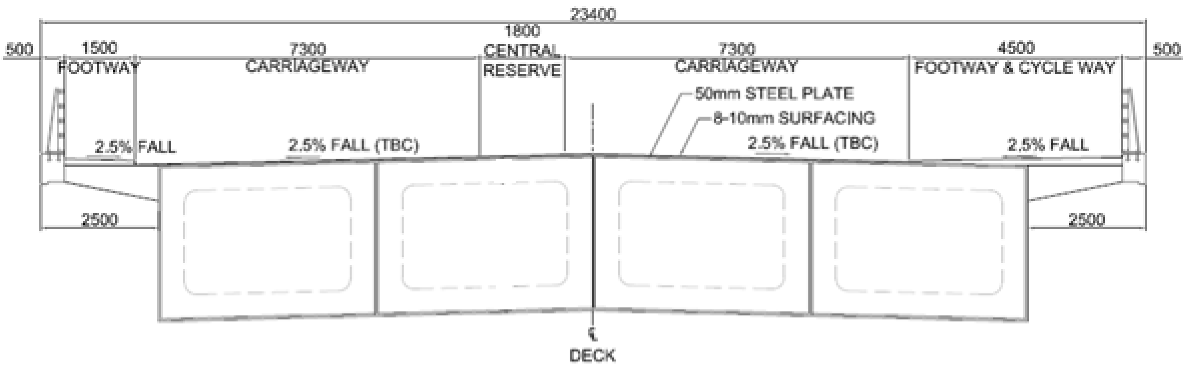


Fig. 4.15 - Box girder arrangement deck (adapted Mouchel/WSP report)

2) 4 No. I beam girder

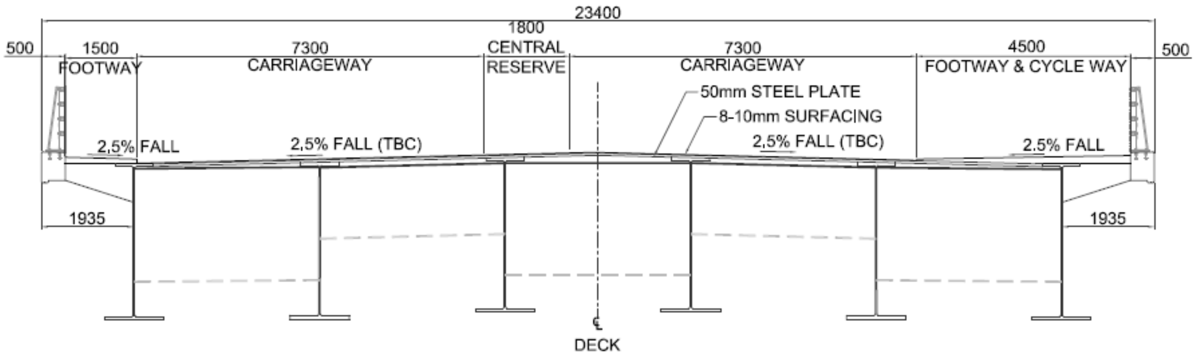


Fig. 4.16 - I beam girder arrangement deck (adapted Mouchel/WSP report)

3) 3 No. trapezoidal box girder

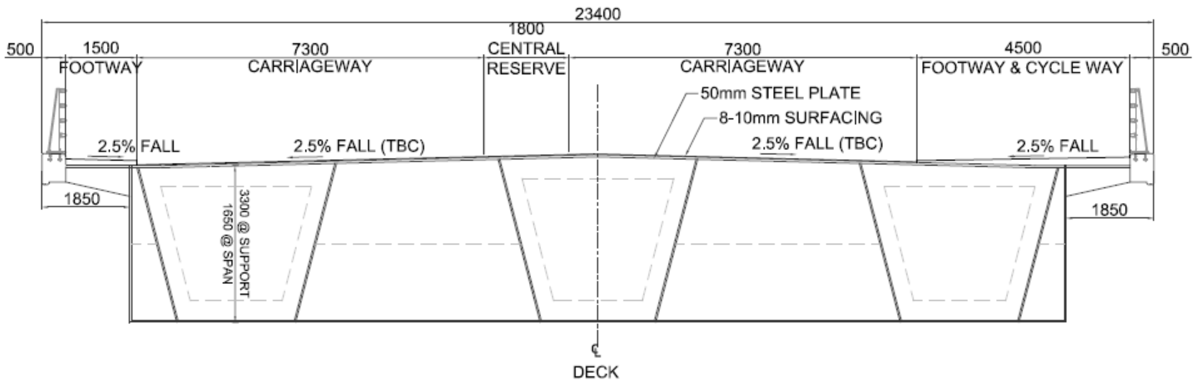


Fig. 4.17 - Trapezoidal box girder arrangement deck (adapted Mouchel/WSP report)

4) 2 No. trapezoidal box girder

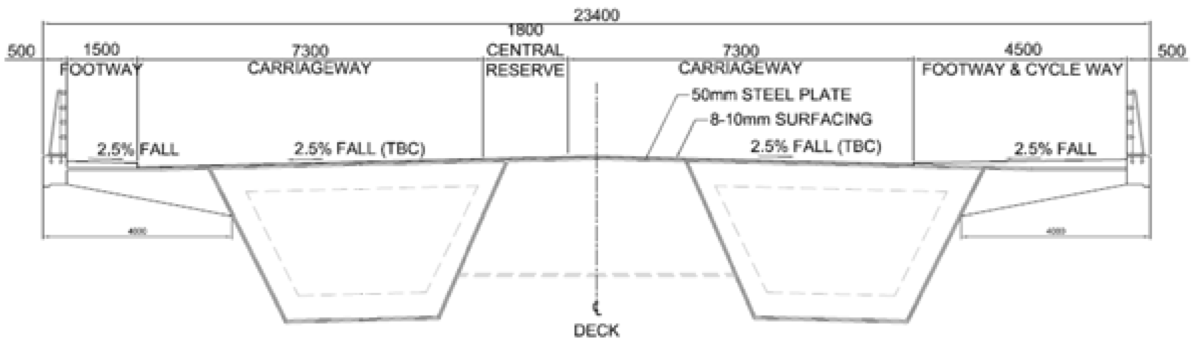


Fig. 4.18 - Trapezoidal box girder arrangement deck (adapted Mouchel/WSP report)

A box girder provides a more rigid layout which has a large torsional stiffness that the deck is subjected to when raised. The torsional stiffness of a closed box section is about approximately between 100 and 1000 times greater than an I beam section. (Kollbrunner, Basler, Glauser, & Johnston, 1969)

The I beams outline, although is simple to design and relatively easy to build, for movable bridges they are subjected to torque and thus more unstable.

The shape of trapezoidal boxes is the ideal in this case because, with this configuration it is possible to reduce the number of boxes and thus the amount of steel required, comparatively with the box shape. Also, this shape facilitates passing of the wind and provides sleek appearance.

The preferred option between the various numbers of trapezoidal boxes is the 3 No. trapezoidal box girders. Though the 2 No. would be best in terms of reducing the number of hydraulic cylinders needed (one per box girder) and thus an easy operation of the bridge, as these must work together as one, the size of the boxes would be too big and unsustainable. Also, in case of a cylinder failure the 3 No. trapezoidal boxes could give a better response with the remaining ones. This option carries the box section intermittently along the length of the leaves, though it changes in dimensions, and three hydraulic cylinders with a maximum hydraulic pressure of 200 bar. It will be required stiffeners (inside the boxes) along the length of the span and a bracing system between the boxes for stability purposes.

4.14.2. Operating Mechanism Layout

As concluded in section 4.12.3 the lifting mechanism more suitable for this type of bridge is hydraulic cylinders. For the various possible arrangements for the cylinders, it has been considered some simple fixed trunnion arrangements. It were not been considered a below-ground Scherzer arrangement as this solution has relatively little benefit.

- **Vertical cylinder arrangement**

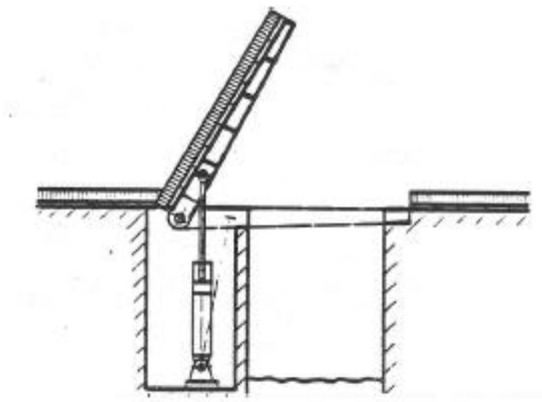
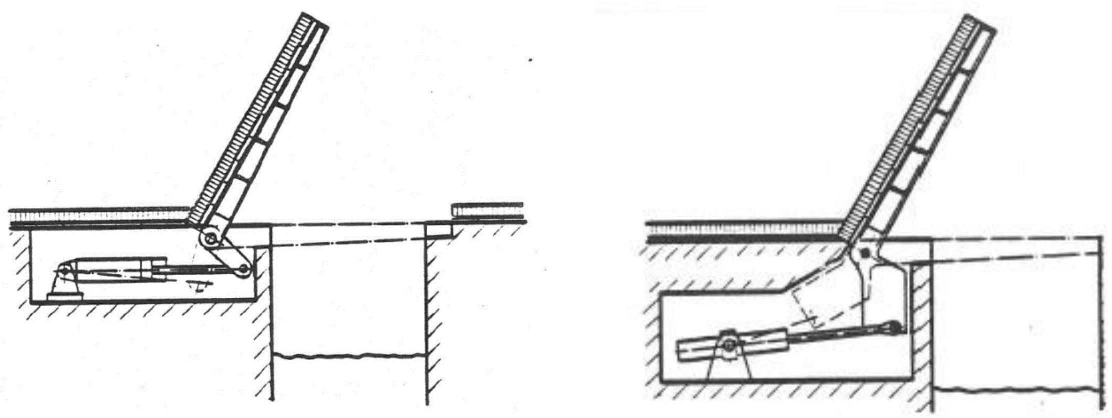


Fig. 4.19 - Bascule bridge with fixed pivot point (KGAL, 2017)

This arrangement has relatively high power but requires a big depth pit for the hydraulic cylinders.

- **Horizontal cylinder arrangement**



(a) (b)

Figure 4.20 - (a) Bascule bridge with fixed pivot (b) Bascule bridge with fixed pivot and counterweight (KGAL, 2017)

Figure 4.20 shoes arrangements of horizontal cylinder layouts without and with the addition of a counterweight, which accomplish a reduced cylinder size and consequently drive power.

The horizontal arrangement would reduce the complexity of construction as it decreases the depth of the pit. But then again, after further investigation, it was found that the cylinders for this kind of arrangement would not be so effective.

- **Another cylinder arrangement**

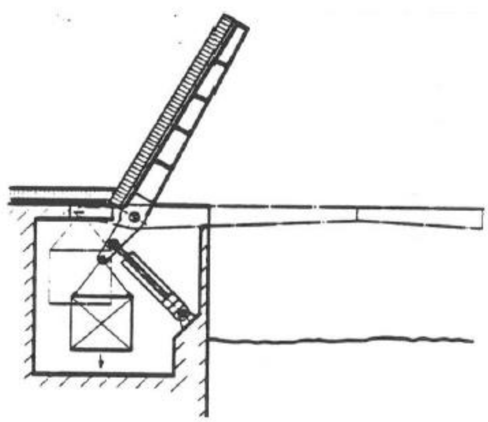


Figure 4.21 - Bascule bridge with fixed pivot and hanging counterweight (KGAL, 2017)

Figure 4.21 shows an arrangement that requires a large pit to house the hanging counterweight and allow it to swing. It would not be the best choice as a big depth pit and construction complexity would be necessary.

As a result, the preferred option is the vertical cylinder arrangement with cylinders in front of the pivot. As the deck comprises a trapezoidal box layout, the possibility of optimizing this option with counterweight placed inside this boxes with concrete and then finishing the need for a pendulum was considered. This would minimize the dead load and reduce the power needed to raise the bridge.

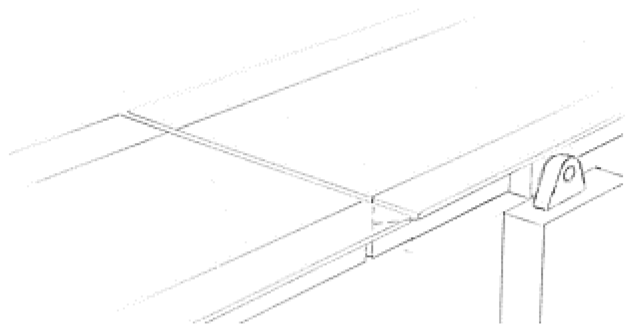
Two challenges in the designing of a bascule bridge involves the lifting process of the bridge. The first challenge is the wind forces when in open position. Since the bridge deck has a large exposed area this led to an assumption requirement of 3 cylinders to capably hold the wind effects. As Port Authority do not want to impose any permanent restriction on the capability of the bridge, the wind loading has been set for full loading according with BS EN 1991-1-4. Note that technically movable bridges are outside the scope of this EC so in a later stage of detailed design, specialist knowledge is required. The second challenge is the occurrence of a failure with the hydraulic cylinders. In a case of a failure, it may be necessary to either raise the bridge or safely return to is close position. As the cylinders are no longer applying a symmetrical load onto the deck, the bridge deck must resist an additional torsion.

It will be necessary to employ PLC (Programmable Logic Controller) controlled proportional control valves which can stabilise the speed of the bridge leaf throughout the whole operation.

4.14.3. Back Span Arrangements

Several back-span arrangements were considered to find the most suitable arrangement between the fixed and moving parts of the bridge, the transverse position of the trunnions relative to the deck and the width and position of the kentledge. The arrangements options are presented below:

- **Option 1 – Straight joint**

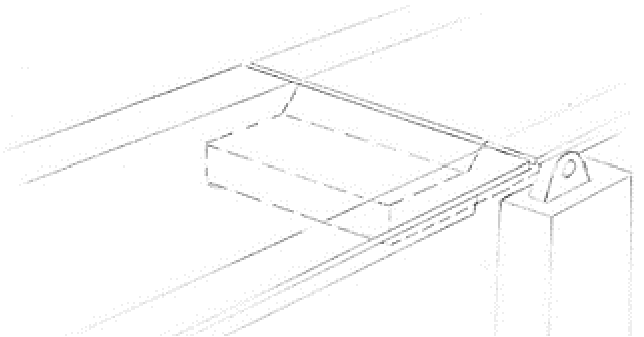


- Simple installation
- No longitudinal joints
- Requires larger piers
- Large bascule pit opening
- Joint over end of span

Fig. 4.22 - Back span straight joint arrangement (KGAL, 2017)

Option 1 has the simplest installation and fewest joints. Therefore, it will present the least influence on maintenance requirements. In this option there is the advantage of the leakage occurring over the edge of the deck. The trunnion is located outboard of the walkways, to avoid clash. However this has the disadvantage of having the necessity of a wider pier to accommodate the trunnion support outside of the parapets, leading to an aesthetic impact. Another disadvantage is when the leaf is raised, the deck comprises a large opening.

- **Option 2 - Straight joint, kentledge below slab**

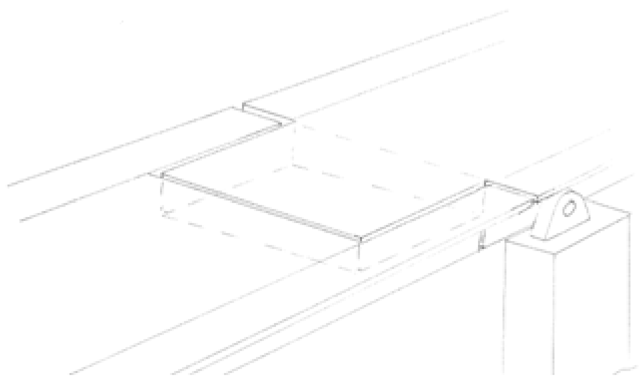


- Complex installation
- No longitudinal joints
- Requires smaller piers
- Small bascule pit opening
- Joint over critical section

Fig. 4.23 - Back span straight joint with kentledge below slab arrangement (KGAL, 2017)

This option arrangement eliminates the problem of having a large opening in the deck when the leaf is raised, but consequently a more complicated construction sequence is needed to allow the kentledge to be manoeuvred below the approach slab.

- **Option 3 - Stepped joint, no overhang**

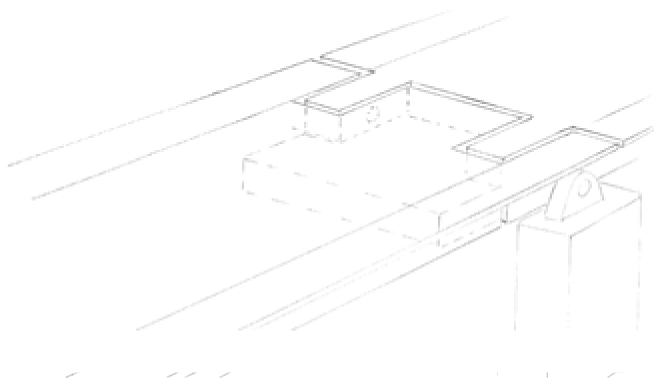


- Longitudinal joint
- Requires smaller piers

Fig. 4.24 - Back span stepped joint arrangement (KGAL, 2017)

In this option the trunnion pier are located underneath the walkways. As the kentledge is located in front of the approach span, longitudinal gaps occurs. However, this can possible be mended by local features. This has the same problem as the option 1 with a large opening when the bridge is raised.

- **Option 4 - Stepped joint , kentledge below slab**



- Complex installation
- Many longitudinal joints

Fig. 4.25 - Back span stepped joint with kentledge below slab arrangement (KGAL, 2017)

As in option 2, this arrangement eliminates the problem of having a large opening in the deck when the leaf is raised, but consequently a more complicated construction sequence is needed to allow the kentledge to be manoeuvred below the approach slab.

The kentledge arrangement was also investigated to find if it would be possible to use a shorter and deeper counterweight with the objective of reducing the required back span and hence reducing the depth of the counterweight pit. It was then determined that the optimum arrangement requires approximately 12m of back span and a relatively narrow kentledge arrangement to allow the back span to remain clear of the cylinders. For this purpose of a large back span, option 1 and 2 are the most advantageous ones.

In conclusion, after detailed consideration and weighing up the advantages of both options, the choice was to use option 1. It is noted that to take into account the above width requirements, (to thicken the section at the trunnion pier), it ended widening the pier and include the trunnion support inside, for purposes of simplification all the section of the pier. It is noted that every longitudinal joint is required to be carefully detailed to prevent causing a hazard to cyclists and pedestrians, however it is not established in the choose option.

4.14.4. Articulation Arrangements

4.14.4.1. Tail and Nose locks

When in closed position, the back span of the bridge will likely be locked with hydraulic tail locking bolts.

A shear lock was considered for the connectivity between the leaves, preventing discontinuities of the carriageways due to the deflections of the bridge leaves, but the rotational equilibrium to live and dead loads must be carried by trunnions and an additional live load support.

4.14.4.2. Live Load Supports

When in closed position, the bascules leaves support live load by means of live load supports which in this case, elastomeric bearings (‘live load shoes’). These must be positioned in front of the trunnions and between the soffit and front wall of the pier. Hence, the calculation carried out assumes that the tail lock moment is carried by the live load shoe.

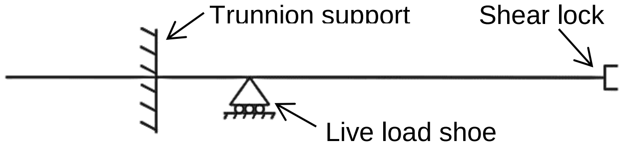


Fig. 4.26 - Diagram of half double leaf trunnion with live load shoes

The transmission of live load into the pier and approach spans is a very delicate feature, so afterwards it is recommended a detailed calculation of these using for example Finite Element Methods.

4.14.5. Substructure and Foundations

The substructure here considered will be the same as discussed before in section 4.9 and 4.10, comprising a pit to house the back span of the leaves and counterweight when in open position. This will be of a reinforced concrete box structure and founded with reinforced concrete piles.

It was forward that the pile tension is not acceptable and the allowed maximum compression is 4000kN per pile, with a 64 no. arrangement of Ø 1200mm. However, this has to be subjected afterward to a more detailed geotechnical study.

4.14.6. Ship Collision Protection

The consideration made was the construction of knuckle walls supporting a fendering steel system along the walls as protection to potential vessel collision. These walls would be founded on piles independent from the piles from the main pier. If with further geotechnical work it will be concluded that these piles do not have suitable capacity, the fenders will have to work together with the main pier itself.

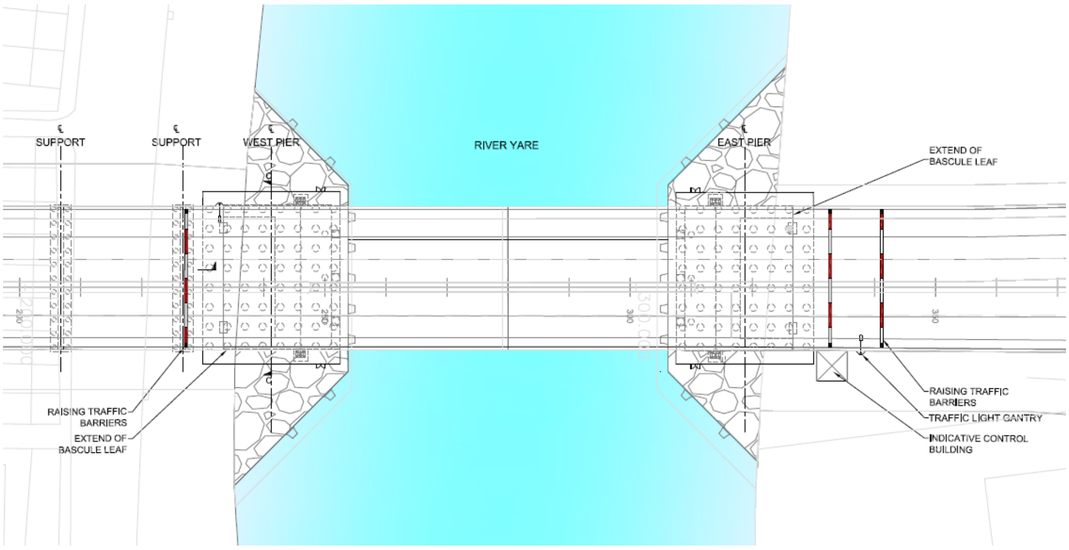


Fig. 4.27 - Plan of the bridge showing the Knuckle walls with fendering steel system (Mouchel/WSP, 2017)

As the knuckle wall will be constructed before the main pier, these foundations have to be piled through the knuckle wall gravel fill. Local checks would be needed to ensure that the capacity of this system is acceptable.

4.14.7. Operation of the Structure

After a detailed navigation study for this exact location, it is estimated the average number of openings for this bridge will be 8-10 times per day.

A hydraulic cylinder's arrangement for lifting the bridge with a power rack of 330kW per leaf was selected. This corresponds to an opening time of approximately 5 minutes and closing time of approximately 3.5 minutes with wind conditions, making it a total cycle time of around 8.5 minutes. With no wind conditions, these times will be reduced to approximately 3.5 minutes to opening and the same to closing, making it a total of around 7 minutes.

Though these times do not include the time needed to pull and drive the shear locks and the time to lower the traffic barriers for vehicular/pedestrian traffic, plus the 10 minutes in advance required by the harbour authorities and discussed in section 4.12.2. In addition, the time is required to clear any vehicle/pedestrian traffic from the bridge itself.

For purposes of a safe operation of the bridge, this control has to be carry out in a suitable place with good visibility. The location of this place - control room - which contains the operating and control mechanisms is yet to be determined, as this is an early stage, nevertheless it is possible to be located above the plant rooms. The bridge shall be operated from this under PLC (Programmable Logic Control) control and the communications with the river users and the control rooms of the other two existing bridges shall be made using radio technology with VHS (Very High Frequency) and landline telephone.

4.14.8. Analysis

For the preliminary design, a number of Eurocodes and UK National Annexes were used. These documents are listed below:

Table 4.4 - Design Documents

Part of Eurocode	Title
BS EN 1990 (Eurocode 0)	Eurocode: Basis of structural design
NA to BS EN 1990	UK National Annex for Eurocode: Basis of structural design
BS EN 1991-1-2 (Eurocode 1)	Traffic Loads on bridges
BS EN 1991-1-1 (Eurocode 1)	Actions on Structures
NA to BS EN 1991-1-1 to 1991-1-7	General actions
NA to BS EN 1991-2	Traffic Loads on bridges
BS EN 1997-1 (Eurocode 7)	General rules

Dead load, superimposed dead load, moving live loads and wind loads were considered in the design. For the dead load, the preferred option described in section 4.14.1 is calculated and allowance for stiffness and connection were also considered.

4.14.8.1. Permanent Loads

Dead load was calculated based on the total area of the deck, regarding the arrangement option choose.

Superimposed dead load is calculated assuming a parapet of 1.40m height and a 10mm surfacing on the bridge deck.

4.14.8.2. Traffic Loads

Live load is calculated using the load models 1 and 3 from UK National Annex to Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges, and applied using the moving load feature in midas Civil with verification done by hand calculations. The groups of traffic load were considered according to the Table NA. 3 (NA to BS EN 1991-2: 2003) and it is shown in Figure 4.30.

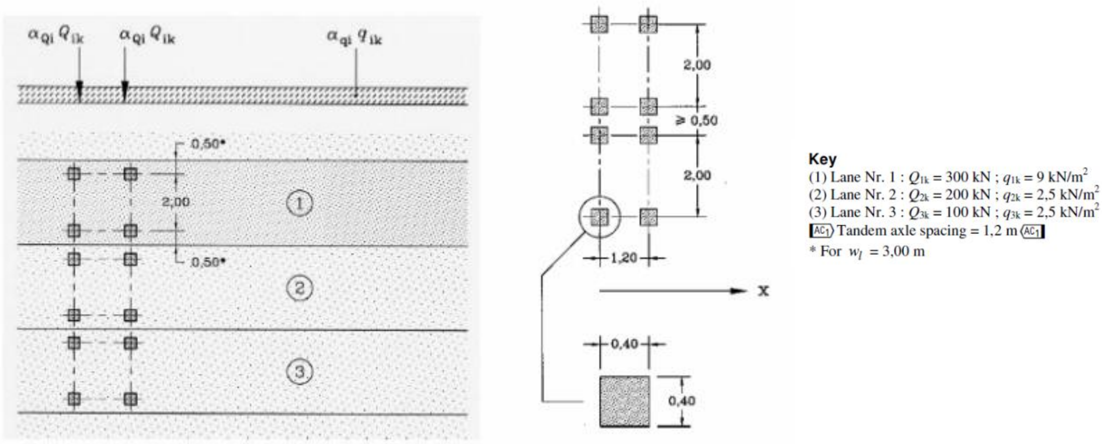
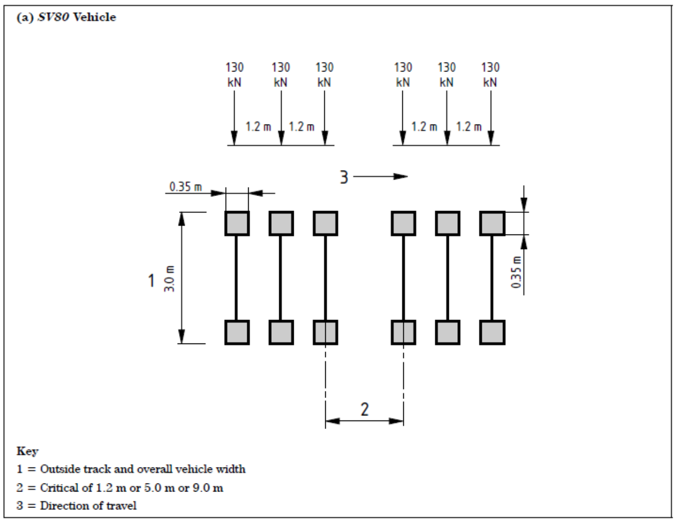
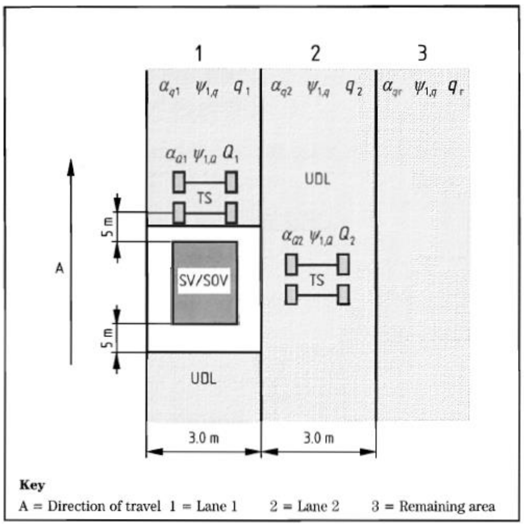


Fig. 4.28 - Application of load model 1 (NA to BS EN 1991-2: 2003)

In order to maximize the load effect at the part of the structure under consideration, the notional lanes are located near the edge of the carriageway with lane no.1.



(a)



(b)

Fig. 4.29 – (a) Configuration for the SV80 vehicle loads for load model 3 (NA.1) (b) Application of SV and Load Model 1 loading when SV vehicle lies within a notional lane (NA to BS EN 1991-2: 2003)

In the absence of more detailed information, complementary load models for special vehicles are assumed for a SV80 vehicle. This is intended to model the effects of STGO Category 2 Vehicles and it has the highest vehicle load with a maximum gross weight of 80 tonnes and a maximum basic axle load of 12,5 tonnes.

Load type		**Carriageway**						**Footways and cycletracks**
		Vertical forces				**Horizontal forces**		**Vertical forces only**
Reference		4.3.2	4.3.3	Annex A	4.3.5	4.4.1	4.4.2	5.3.2.1 Equation (5.1)
Load system		LM1 (TS and UDL)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces	Centrifugal and transverse forces	Uniformly distributed load
Groups of loads	gr1a	Characteristic						0.6 times Characteristic
	gr1b		Characteristic					
	gr2	Frequent[4]				Characteristic	Characteristic	
	gr3[1]							Characteristic
	gr4				Characteristic			Characteristic
	gr5	Frequent[4]		Characteristic				
	gr6			Characteristic		Characteristic	Characteristic	
		Dominant component action (the group is sometimes designated by this component for convenience).						

(1) This group is irrelevant if gr4 is considered
(2) Characteristic value obtained from 5.3.2.1
(3) This is a reduced value accompanying the characteristic value of LM1 and should not be factored by ψ_1. However, when gr1a is combined with leading non-traffic actions this value should be factored by ψ_0
(4) The ψ_1 factors should be taken from the UK National Annex to BS EN 1990

Fig. 4.30 - Assessment of groups of traffic loads (NA to BS EN 1991-2: 2003)

Longitudinal live load is considered through NA.2.17, therefore is equal to 900kN.

NA.2.17 Upper limit of the braking force on road bridges [BS EN 1991-2:2003, 4.4.1 (2)]

The upper limit for the braking force should be taken as 900 kN.

Fig. 4.31 - Longitudinal load (NA to BS EN 1991-2: 2003)

4.14.8.3. Wind

Wind load is anticipated to be critical when the bridge is opened. This will govern the size of the cylinders that will be used to operate the bridge. While the bridge is closed three different type of winds which are vertical, transverse and longitudinal, are considered in accordance to EN1991-1-4 & NA.

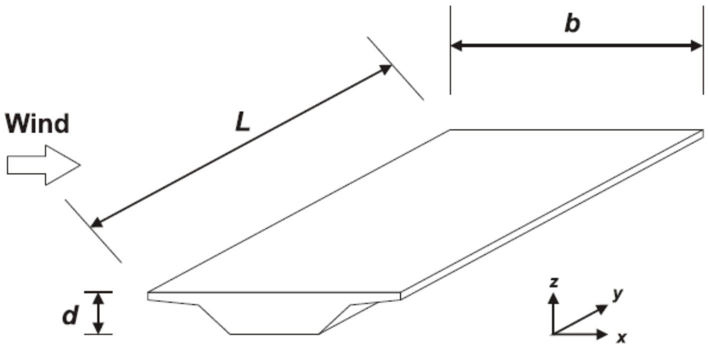


Fig. 4.32 – Directions of wind actions on bridges (BS EN 1991-1-4-2005

All the physiognomies that have to be taken into account in the wind characterization and all the steps carried out can be found at appendix A.

Table 4.5 – Results of wind calculations

		Force Applied (kN)	Bending Moment M (kN.m)	Shear V (kN)
Bridge lifted	Vertical wind	51.5	28754	1751
	Longitudinal wind	3.9	-	-
	Transverse wind	6.3	-	-
Bridge closed (in service)	Vertical wind	21.1	12206	718
	Longitudinal wind	3.9	26	3.2
	Transverse wind	6.3	3642[6]	214

[6] This moment is in another direction - transverse wind force causes torsion in the deck.

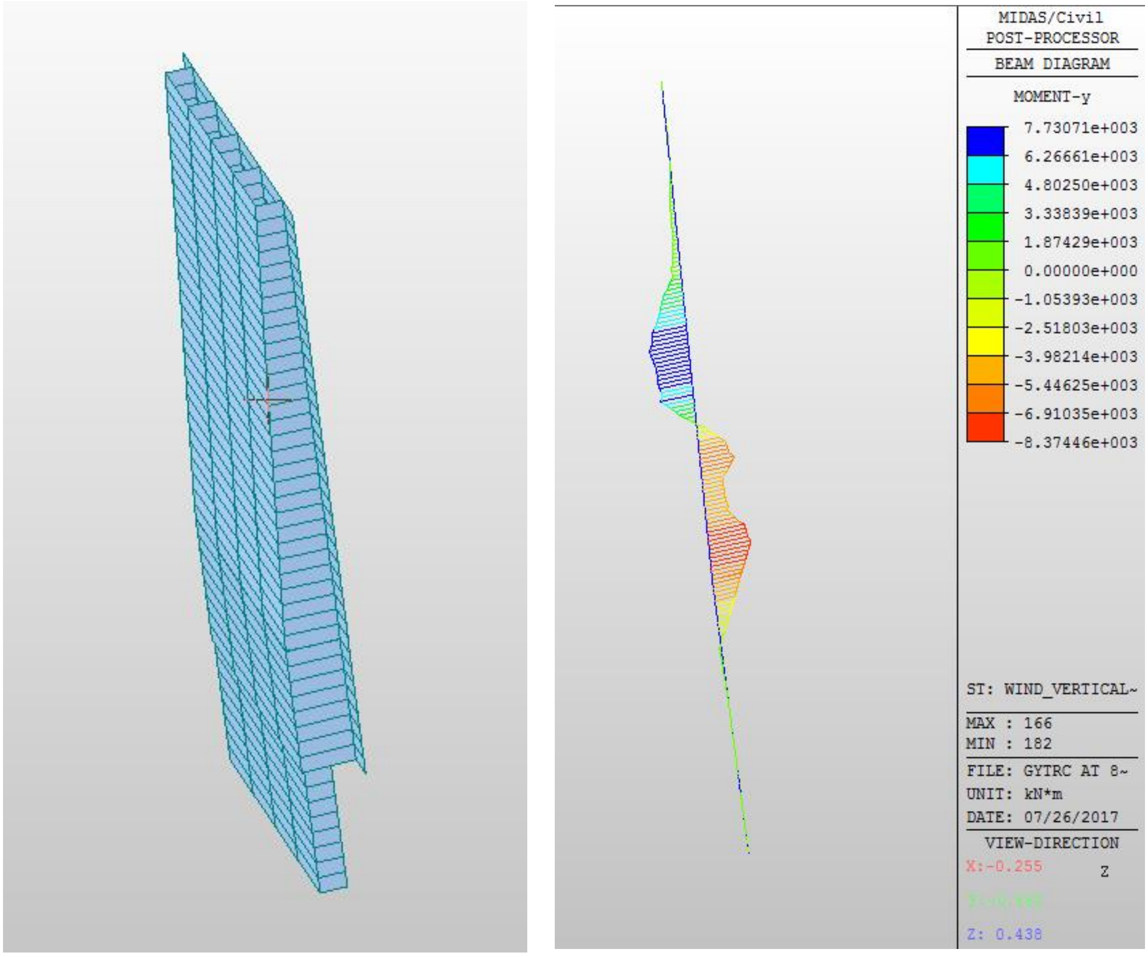


Fig. 4.33 – Results of bending diagram (Mouchel/WSP, 2017)

4.14.8.4. Snow

UK National Annex does not contain specific guidance on γ for snow loading as it can hardly be ignored for UK bridges. However, it needs to be checked for a movable bridge. Consider factor from EN1990 as 1.5 'For all other variable actions'.

$$s_k = [0{,}15 + (0{,}1Z + 0{,}05)] + \left(\frac{A - 100}{525}\right) \quad \text{(NA.1)}$$

where

s_k is the characteristic ground snow load (kN/m^2);

Z is the zone number obtained from the map in Figure NA.1;

A is the site altitude (m).

Fig. 4.34 – Equation NA.1 of characteristic ground snow (NA to BS EN 1991-1-3-2003)

As this bridge will be located in the UK and the results of the snow calculations are not that significant (table 4.6) in this case, it was then decided not to apply snow load.

Table 4.6 – Results of snow calculations

	Shear (kN)	**Bending Moment (kN.m)**
Bridge closed	531	5836

4.14.8.5. Combinations and Load Factors

Through the BS EN 1991-1-1 load combinations used in road bridges, these fundamental combination of actions (Eq. 4.1) are used to perform ULS safety verifications on shear and bending strength capacity of cross-sections.

$$Ed = \Sigma\gamma_G G_k + \gamma_P P_k + \gamma_Q Q_k + \Sigma\gamma_G \Psi_0\, Q_k \tag{4.1}$$

The factors applied to the loads are described in the tables below, in accordance to Tables NA.A2.1 and Tables NA.A2.4 of NA to BS EN 1990 (appended):

Table 4.7 - Longitudinal load

	Ψ_0	ψ_1	γ EQU (A)	γ STR/GEO (B)	γ STR/GEO (C)
Traffic (gr1a or gr5) as lead	0.75	0.75	1.35	1.35	1.15
Traffic gr1 as accomp.	0.75	0.75	1.35	1.35	1.15
Wind as lead	1	1	1.7	1.7	1.45
Wind as accomp.	0.5	0.2	1.7	1.7	1.45
Snow as lead	0.8	1	1.5	1.5	1.3

Table 4.8 - Longitudinal breaking/acceleration forces

	ψ1	γ EQU (A)	γ STR/GEO (B)	γ STR/GEO (C)
Traffic (gr2), ψ already applied[7]	-	1.35	1.35	1.15
Breaking	1	1.35	1.35	1.15

Through the Tables NA.A1.2 – Design values of actions, annexed to this work, Ultimate Limit State load combinations were considered during different position of the bridge.

In service (closed position) traffic load governs as leading variable, wind as accompanying. It was considered the worst of traffic gr1a and gr5 (STR/GEO) (Set B):

[7] to apply this forces need to reduce vertical loading of provided gr2 loading to 'frequent' (ψ= 0.75)

Combination 1 → gr1a: LM1 "+" Pedestrian

1.2xDL + 1.35x[(TS+UDL) + (0.6xPedestrian] + 1.7x0.2xWind

Combination 2 → gr5: LM3 "+" LM1 (frequent)

1.2xDL + 1.35x[SV80 + 0.75x(TS+UDL)[8]] + 1.7x0.2xWind

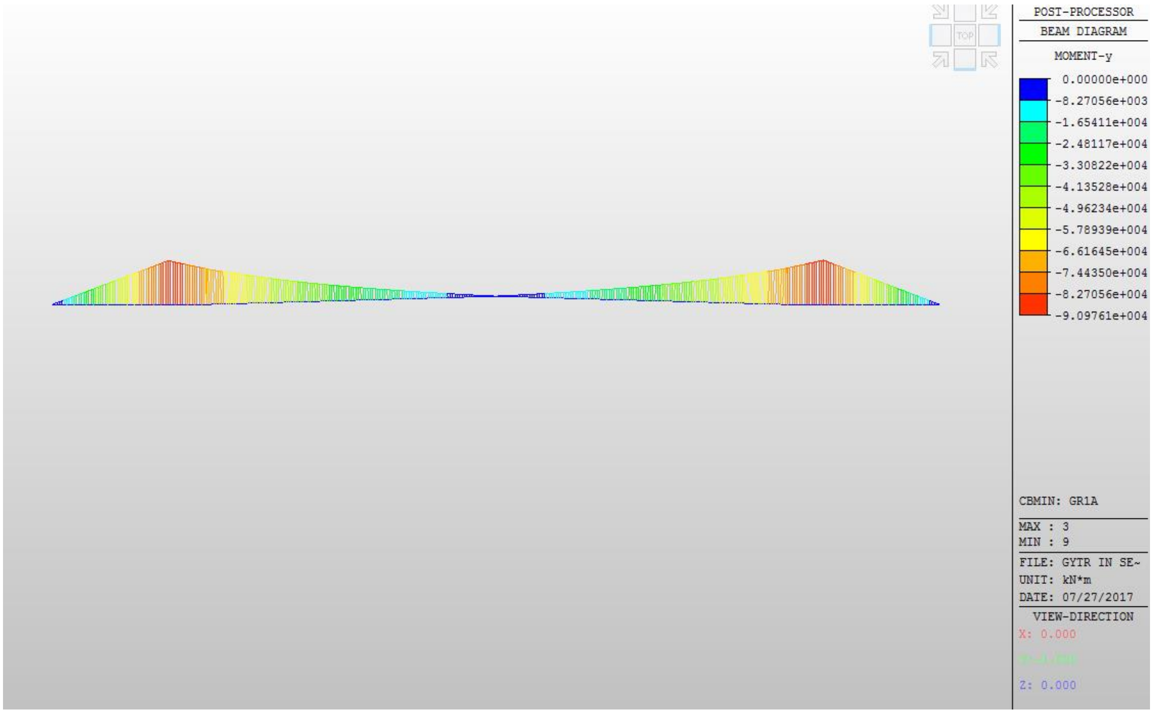


Fig. 4.35 – Results of combination 1 (Mouchel/WSP, 2017)

In lifted position (opened position) wind governs as leading variable and there are no accompanying loads:

Combination 3 → 1.7xWind + 0.95xDL

In lifted position the dead load is favourable to the operation and it is then considered in the calculations with a favourable factor of 0.95.

Table 4.9 – Results of load combinations

		Shear (kN)	Bending Moment (kN.m)
Bridge closed	Combination 1	15533	290328
	Combination 2	14418	277492
Bridge lifted	Combination 3	7061	41683

[8] LM1 according to gr5

4.14.8.6. Preliminary Calculations

Preliminary calculations were carried out using hand calculations and computer modelling using midas Civil and Autodesk Structural Bridge Design. The structure capacity of the deck, pier and foundation were carried out using hand calculations and validated using Autodesk Structural Bridge Design. The load effect calculation were carried out using hand calculations and midas Civil. A simple line beam structure was used for the modelling.

Hand calculations are based on an un-propped cantilever representing a single bascule leaf. This assumption is slightly conservative, since the shear link at the bascule noses makes each span behaviour somewhere between a cantilever and a propped cantilever – elastically propped cantilever.

To estimate the axial force in the foundation (piles), the reactions from the superstructure model and the hand calculations were compared, validated and used for the foundation design.

Detailed design would require a much more refined model.

All the calculations that were carried out can be found in appendix A.

4.14.8.7. Model

The two leaf bascule bridge is analysed using a two dimensional (line beam) linear structural static analysis computer software midas Civil considering the geometry, loads and materials. Since the superstructure (both leaves) is symmetric, only one leaf would be enough to model. However, it was carried out two separately models, for only one leaf and the whole structure. The purpose of these two models was to see if there was any variations in the results to verify the assumption that an unpropped cantilever is reasonable.

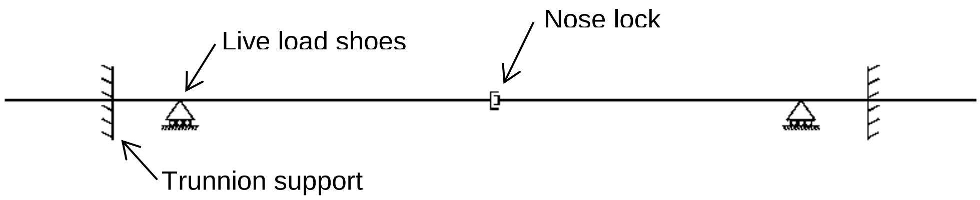


Fig. 4.36 - 2D Hand model

For the analysis, the bridge can be schematised as in Figure 4.36.

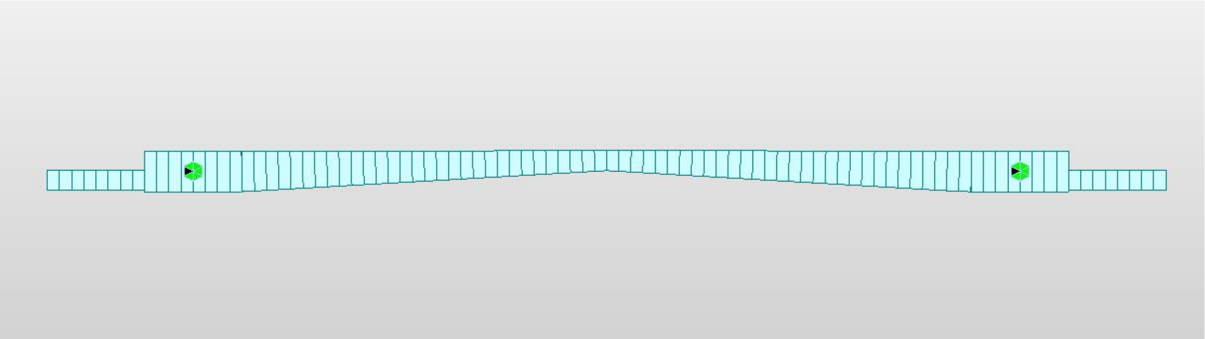


Fig. 4.37 - 2D midas Civil model (Mouchel/WSP, 2017)

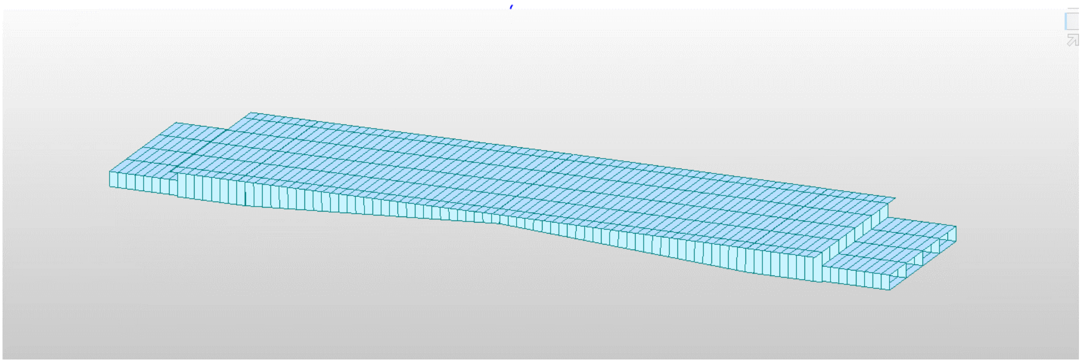


Fig. 4.38 – Model of whole bridge in 3D view (Mouchel/WSP, 2017)

The boundary conditions for the leaf in the model assumed to provide additional vertical-only support (Dz) for the live load shoes and the pivots supports fixed in rotation about the longitudinal axis (Rx), allowing rotation about the transverse axis (Ry).

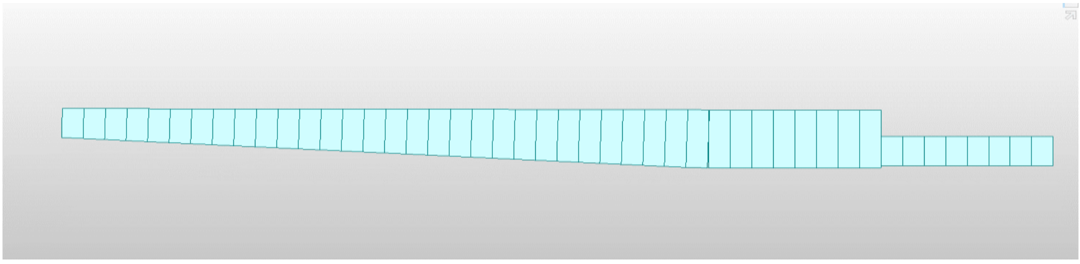


Fig. 4.39 - Model of half bridge (Mouchel/WSP, 2017)

For the model of the whole superstructure, the boundary conditions at the pivot points were fixed in Dx, Dy, Dz, Rx, Rz and at tail locks Dz was fixed. The nose locks of the leaves as they only transmit a vertical shear, are modelled as a beam end release, allowing Ry rotation and longitudinal displacement (Dx).

4.14.8.8. Capacity Calculations

For purposes of initial sizing of the structure it was carried out preliminary capacity calculations. The following materials and properties were assumed for the bridge structure components:

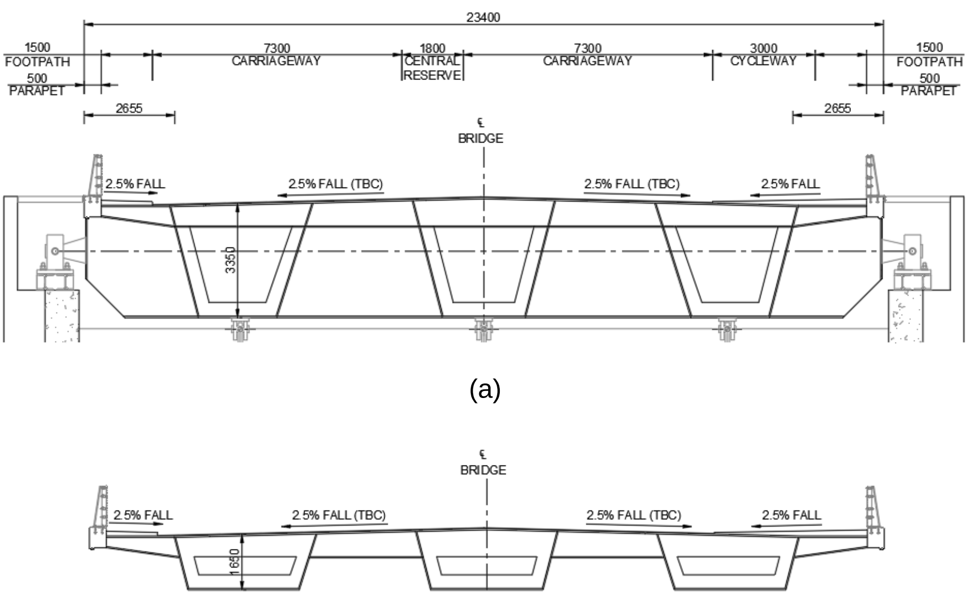
Structural Steel – fy = 355 MPa

Concrete C40/50 – f_{ck} = 40 MPa

Situation is entirely hogging in service. Sagging will only apply to horizontal wind on lifted bridge. These calculations can be seen in appendix A, which can be summarise:

Table 4.10 – Summary of load cases (ULS) for the deck at support

Combinations	**Hogging**							
	M_{Ed} (kNm)	**M_{Rd}** (kNm)	**M_{Ed}/M_{Rd}**		**V_{Ed}** (kN)	**V_{Rd}** (kN)	**V_{Ed}/V_{Rd}**	
1	290328	664887	0.44	**PASS**	15533	125212	0.12	**PASS**
2	277492	664887	0.42	**PASS**	14418	125212	0.12	**PASS**
	Sagging							
3	41683	304045	0.14	**PASS**	7061	125212	0.06	**PASS**



(a)

(b)

Fig. 4.40 - Final deck arrangement: (a) At Support (b) At Mid-span (Mouchel/WSP, 2017)

The final sizes are described in Table 4.11.

Table 4.11 – Final sizes

Final sizes	Values (mm)
Height of box	3300
Box width	4200
Width of the top plate and edge cantilevers	50
Width of the bottom plate	50
Width of the web	35
Transverse stiffener spacing	2000

As it can be seen in table 4.10 the factors are a little low, with both bending moments and shear passing by a wide range of safety. We could reduce the width but the reason behind the choice of maintaining this sizes is the deformability of the deck in service ultimate state.

To attest the serviceability it was carried out a simple calculation of the deformability of the span deflection with the final sizes. It was then compared to the deflection of the Midas model.

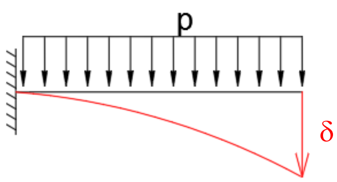


Fig. 4.41 - Displacement of a cantilever with a distributed force

$$\delta = \frac{1}{8}\frac{p.l^4}{E.I} \tag{4.2}$$

The final deflection, calculated conservatively with a cantilever model, result of approximately 35mm. The load considered was the traffic load and dead weight for SLS, which are the most critical in closed position.

AASHTO LRFD design specifications advises that the maximum deformation of a bridge should not exceed $\frac{l}{1000}$, for road bridges with pedestrian traffic. (Barker, Staebler, & Barth, 2011) In this case the maximum deformation is $\frac{56.5}{1000} = 56.5mm$.

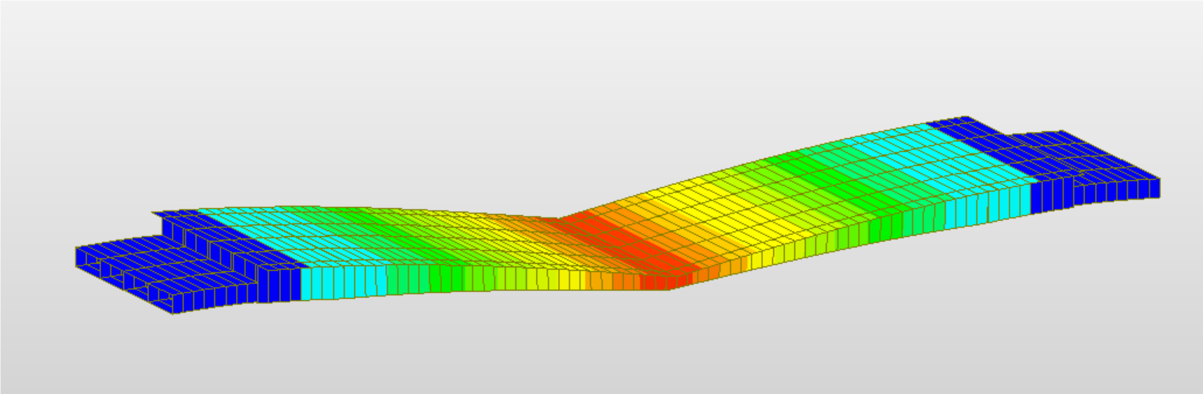


Fig. 4.42 - Displacement of the bridge model (exaggerated) (Mouchel/WSP, 2017)

4.14.8.9. Pier and Foundations

Design of the pier's back wall is very critical due to earth pressure, hence design of this wall was considered to be significant.

Active earth pressures (K_a, γ, h) are considered to ensure that the abutment is stable.

Table 4.12 – Summary of capacity check for the pier at and top of haunch

Haunch							
M_{Ed} (kNm)	**M_{Rd}** (kNm)	**M_{Ed}/M_{Rd}**		**V_{Ed}** (kN)	**V_{Rd}** (kN)	**V_{Ed}/V_{Rd}**	
4815	8695	0.55	**PASS**	-	-	-	**-**
Top haunch							
3956	6363	0.62	**PASS**	800.4	791.5	1.01	**KO**

For the resistance capacity of the wall it was considered the active soil, surcharge and traffic load (as well as the longitudinal).

The shear capacity here calculated does not include shear links, so it is normal that does not pass the capacity check. So it is required to add shear links which was calculated using Autodesk Structural Bridge Design:

- Longitudinal steel with 2 layers ϕ32 diameter.

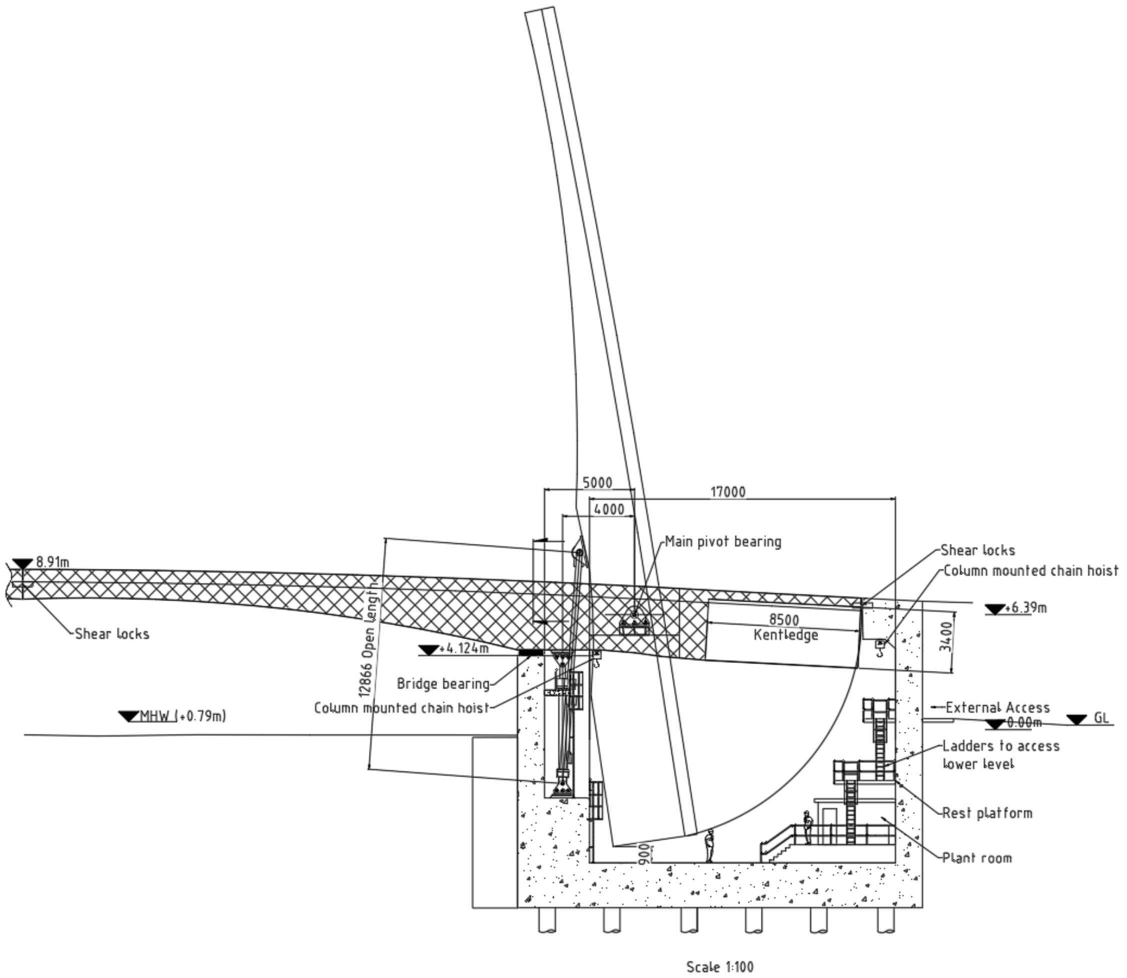


Fig. 4.43 – Main dimensions of the main pier (adapted KGAL, 2017)

The piled design would comprise bored piles.

The spacing of the piles should be greater than approximately 3m, for the reason that less than this reduces the efficiency of the piles.

For purposes of initial preliminary sizing, it is known some indicative and conservative values:

- 1m pile diameter can support approximately between 1500kN and 2000 kN
- 1.5m pile diameter can support approximately between 1800kN and 3000kN

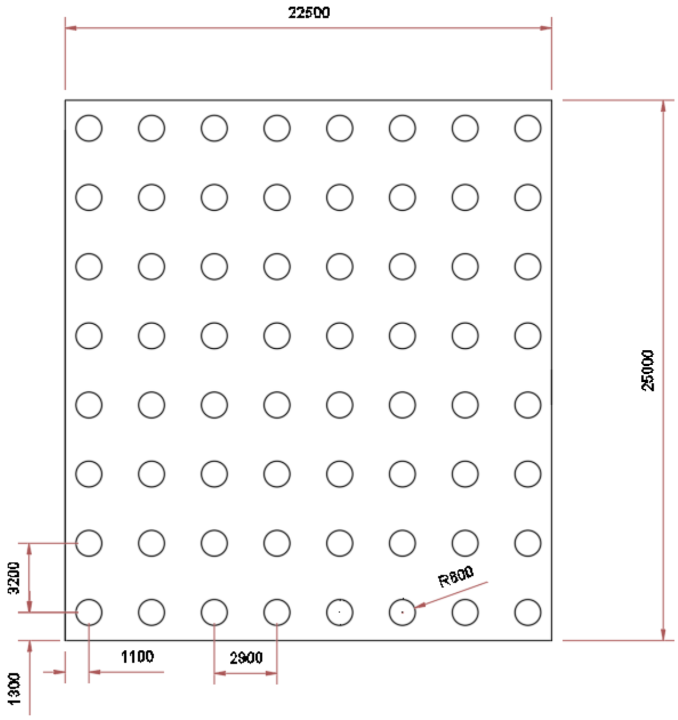


Fig. 4.44 – Pile dimensions of the main pier (Mouchel/WSP, 2017)

It was assumed some dimensions for the pile cap and spacing and then calculated the each pile capacity. For pile groups the distribution of the action by pile is given by:

$$Ni = \frac{N}{n} + \frac{Mx}{\Sigma yi^2}.y + \frac{My}{\Sigma xi^2}.x \tag{4.3}$$

A simplified hand calculations was carried out to compare and attest the viability of this method. With a 20m deep the value of the N_{SPT} taken from the soil tests is 22, so the value of the shear strength (c_u) is 150kPa. Due to the high water level is this location, this value reduces to half.

$$Qs = \alpha.Cu.As \tag{4.4}$$

$$Qb = qb.Ab \tag{4.5}$$

With these, the capacity of each pile can be calculated with equation…

$$Q_T = \frac{Q_s}{Fs} + \frac{Q_b}{Fs} \tag{4.6}$$

To consider the efficiency of the piles group is enough to multiply for a ratio of 0.7 for calcareous soils.

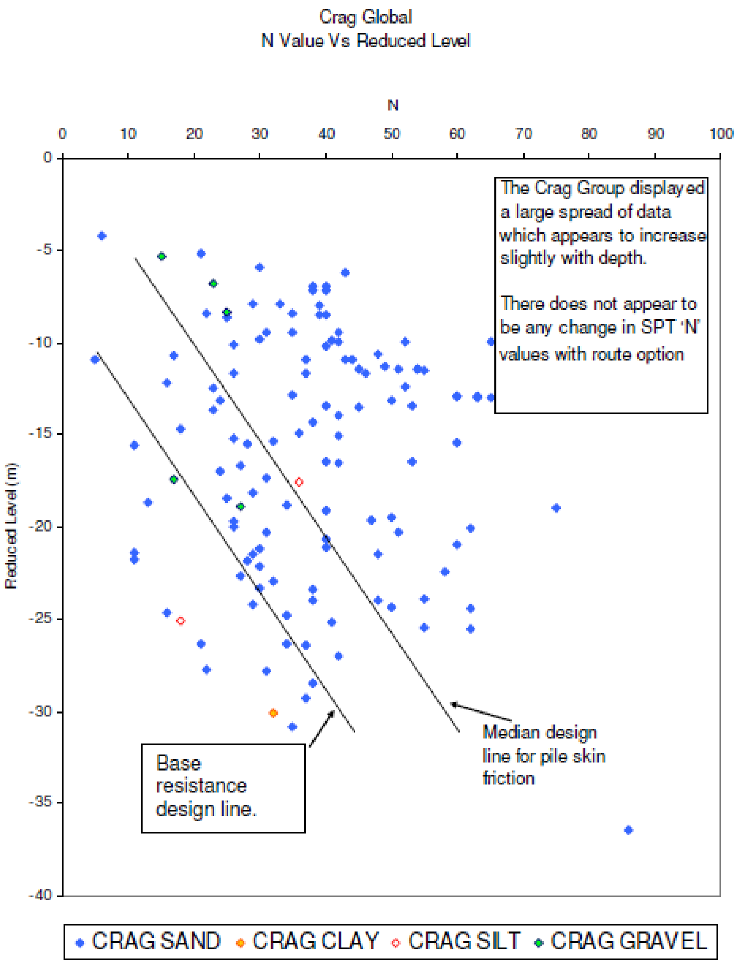


Fig. 4.45 – Design values for crag formation (Mott McDonald, 2008)

4.14.9. BUILDABILITY

In this case, as the fendering system will be incorporated along the knuckle wall, it was decided for purposes of buildability that the construction of this wall be done firstly. It will be needed a temporary enclosure built across the water and pump the water of the enclosed area – cofferdam. This consists in sheet steel piles, providing an effective seal to cofferdam and the new pier construction could be continued inside the cofferdam with no effect on the river water.

For the construction of the orthotropic steel deck it is assumed that the segments are trial assembled on the ground to the same alignment as the final position of the site or nearby yard, to help in later lifting by reducing the lifting distance. Through transportations limits of 4.3m wide and 27.4m long, the assembling of the span will require boxes to be spliced on site and a minimum of 2 sections per cantilever span. In this case, will be transported 3 sections that correspond to the 3 sections boxes of the deck.

Usually, temporary supports, are used mainly in the middle of the channel to help set the leaf into the intended alignment without prejudicing the loads of the deck. After the installation of the operating system and leaf has taken place, the leaf is often set in fully open position. A locking device will be used for this, so the operating mechanism is not under influence of the wind and work can still be done until the bridge will be taken into operation.

The channel is inaccessible during the period of time that the leaves are installed. This period of time is often planned far ahead and cannot be postponed without consequences.

With the deck complete, operations can begin to install the electrical/mechanical systems, roadway barriers, deck water proofing, wearing surfaces, etc.

4.14.10. Maintenance and Inspection

When the bridge is in closed position, the dead and live loads are transmitted to the bridge trunnions, bearings and locks, so it is reasonable to carry a maintenance or inspection to M&E equipment in this period. Nonetheless, with this, the navigation traffic may be affected, so it is required a good planning of the maintenance period times. This plan is covered in the O&M manual, which is to be produced by M&E engineers and is out of scope at this study.

The O&M manual shall contain a detailed description and full instructions for the operation, maintenance and inspection of the bridge. This manual shall also contain measures for both navigation and road/pedestrian traffic during this periods.

All internal parts of the structure should be accessible and so it may be considered construction of permanent platforms and/or stairs to allow for maintenance and inspection, without affecting the normal function of the bridge.

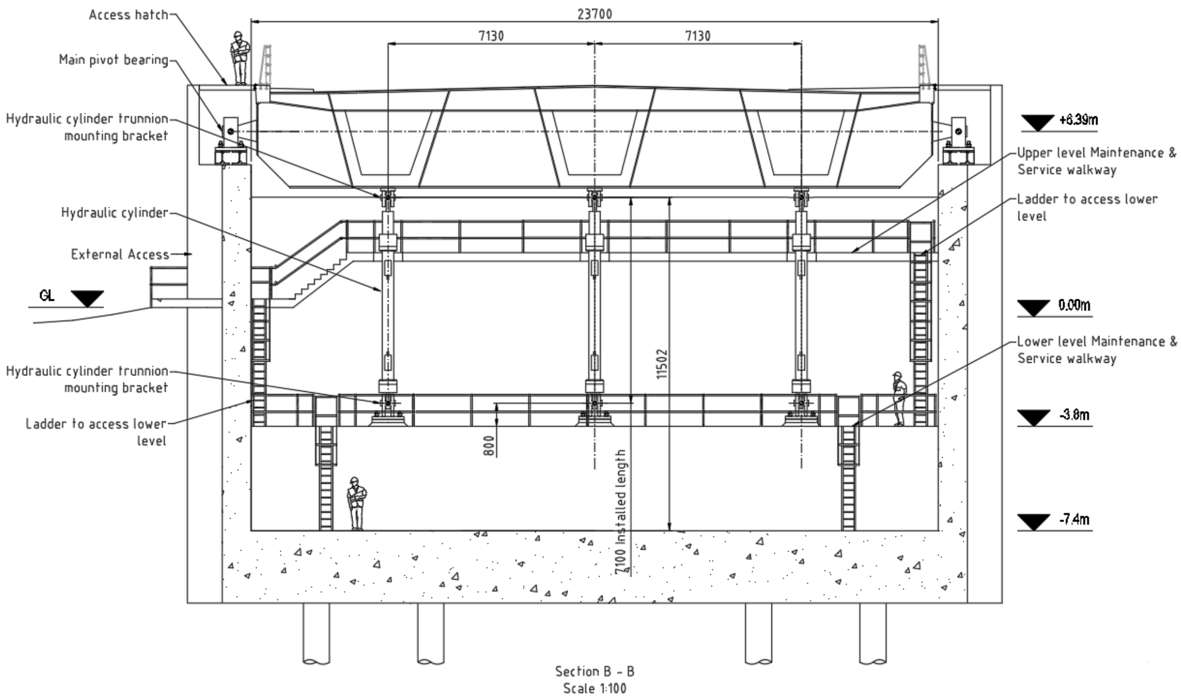


Fig. 4.46 - Interior of pit house (adapted Mouchel/WSP, 2017)

5

CONCLUSIONS AND FUTURE DEVELOPMENTS

5.1. CONCLUSIONS

Movable bridges have been an important part of the transportation infrastructure for centuries. They present unique challenges to the structural engineer and require extensive coordination of the different disciplines systems throughout the whole process design, to achieve a durable and operationally reliable structure.

There have been, for the last decades, advances concerning innovations with new materials, construction techniques and methods of analysis. This need for innovation came from the desire for bridges with more pleasing appearances and it is still the main concern with movable bridges design. The main achieved developments in the last decades comprise a trend for greater use of hydraulic components and lighter deck materials, such as aluminium and orthotropic steel. Nonetheless, this evolution still has a large way ahead. The understanding of the operating mechanisms and their optimisation regarding the bridge opening time are a key point that needs further development. Also one of the most important aspects that is still currently subject of much discussion is the correlation between the structural behaviour and the machinery systems. The types and location of certain and specific machinery completely changes the structural behaviour and balance of the structure and consequently the whole operation.

One aspect which is of extreme importance and usually neglected in the first stages of the design is the buildability of the project. For a good performance of the bridge a suitable construction is necessary, and what works in one location does not work in another. If the design team comes with a project that is not feasible, due to for example transportation and locations constrains or installation of the elements, the consequences can be unbearable.

Movable bridges are much more susceptible to damage due to their complex movable mechanisms and slender structural elements. Hence, the deterioration of the various elements occurs faster and more visible than for fixed bridges. They require constant maintenance to keep up a satisfactory performance, which translates in higher costs compared with those for fixed bridges. Even with the best possible maintenance work, it is still frequent that malfunctions occur due to the many level of components working, which cause interruptions to either the navigational or vehicle traffic.

Regarding the presented case study, for the selected option – Bascule Bridge – the most important aspect considered is the structural stability and reliability of the deck. All the other considerations were carried out with that in thought. The structure meets all verifications established by the Eurocodes and National

Annexes, considering an extreme care taken in the design of constructive details. If the design is taken through a further detailed stage and afterwards properly constructed and maintained, a long service life is likely to be expected.

After the analysis and preliminary calculations, it is possible to assert that there is lack of information, guidelines and especially codes/standards specifications for the particular case of movable bridges and their components. Most of the analysis carried out within this study were based on a combination of American manuals and recommendations for movable bridges – AASHTO – and Eurocodes. In Europe, only the Netherlands and Germany have adapted standards or codes specifically addressing the concept and design of movable bridges. Most countries in Europe, including the UK, do not have specific regulation for this type of bridges.

5.2. Future Developments

The ability of designing a viable structure with a pleasing appearance is perhaps one of the greatest achievements and concurrently a big challenge. Moveable bridges are a very particular case of this. There are still a fair number of developments that have been addressed for this matter, especially in the type of moveable bridges with features markedly above the road level. Bascule and lift bridges would probably require new approaches to address this issue. Counterweight layouts in bascule bridges and imponent towers in lift bridges are the most evident features with impact on aesthetics. A closer cooperation with mechanical & electrical engineering teams and possibly architects could be an important added value to develop new forms of design.

Perhaps the most challenging aspect presented in the current work was how to connect the leaves of the bascule bridge at mid-span point without affecting the behaviour of the overall structure. The modelling of this connection of the bridge entails some particularities that still cannot translate the real behaviour of the bridge. To contour this problem, two different structural models are usually carried out. An analogy model and another with a more conservative but simpler approach model. In the end, the results of both models are compared to confirm feasibility.

The knowledge of the machinery systems and their correlation with the operating mechanism, still generates many doubts, especially for the live load bearings design. The correct type of live load bearings has to take into account the best location and operating mechanism for an optimisation of the pier dimensions and reduction of the moments taken by the rotation support, in this case the trunnion.

REFERENCES

(4 de October de 2014). Obtido de Royal HaskoningDHV: http://haskoning-latam.com/ingles/news-information/Paper-Strengthening-the-Orthotropic-Steel-Deck-Structure-of-the-Movable-Bridge-across-the-Hartelkanaal-The-Netherlands

Abrahams, M. J. (2000). Movable Bridges. Em W.-F. Chen, & L. Duan, *Bridge Engineering Handbook.* CRC Press LLC.

Aluminium Association of Canada. (s.d.). Obtido de Aluminium: https://aluminium.ca/en/aluminium/infrastructure-and-bridges

Barker, M. G., Staebler, J., & Barth, K. E. (2011). *SERVICEABILITY LIMITS AND ECONOMICAL STEEL.*

Berger, I., Healy, D., & Tilley, M. (2015). *Movable Span Bridge Study.*

Birnstiel, C., Bowden, W., & Foerster, G. (2015). *Movable Bridge Design.* ICE Publishing.

Coates, A., & Bluni, S. (2004). Movable Bridge Design . *STRUCTURE magazine*, 4.

Hall, L. (Unknown). *Draw Bridge*. Obtido de http://www.madehow.com/Volume-6/Draw-Bridge.html

Hool, G. A., & Kinne, W. S. (1943). *Movable and long-span steel bridges.* New York: McGraw.

Hovey, O. E. (1926). *Movable Bridges.* New York: John Wiley & Sons, Inc.

KGAL. (2017). *Mechanical, Electrical, Hydraulic and Structural Interface Aspects.*

Knott, M. A. (1990). *VESSEL COLLISION DESIGN OF MOVABLE BRIDGES.*

Koglin, T. L. (2003). *Movable Bridge Engineering.* New Jersey: John Wiley & Sons, Inc.

Koglin, T. L., & Colker, S. (1995). Stabilization of Double Leaf Bascules. *e-periodica*, 7.

Kollbrunner, C. F., Basler, K., Glauser, E., & Johnston, B. G. (1969). *Torsion in Structures.* Springer-Verlag Berlin Heidelberg .

Koutsarsky, A. (19 de September de 2012).

Lane, T. (October de 2016). *South Park Bascule Bridge Replacement*. Obtido de STRUCTURE magazine: http://www.structuremag.org/?p=10487

Mahmoud, K. M. (2003). *Recent Developments in Bridge Enginnering.* New York: A.A BALKEMA PUBLISHERS.

Mike. (1 de January de 2004). *Maritime Journal Insight for the european commercial marine business*. Obtido de East Port Great Yarmouth Awaits Key Decisions: http://www.maritimejournal.com/news101/marine-civils/port,-harbour-and-marine-construction/east_port_great_yarmouth_awaits_key_decisions

Mott McDonald. (2008). *Geotechnical and Geoenvironmental Interpretative Report.*

Mott McDonald. (2009). *Structural Options Working Paper.*

Mouchel/WSP. (s.d.).

Nedev, G., & Khan, U. (2011). *Guidelines for conceptual design of short-span bridges.* Master's Thesis. Department of Structural Engineering, Chalmers University of Technology.

O'Brien, E. J., & Keogh, D. L. (1999). *Bridge Deck Analysis.* London: E & FN Spon.

Ryall, M. J., Parke, G. A., & Harding, P. J. (2000). *The Manual of Bridge Engineering.* Thomas Telford Ltd.

Sloan, A. K. (July de 2004). *The Movable Bridges of the King Bridge Company*. Obtido de http://s174434611.onlinehome.us/moveable_bridges_to_come.htm

Susoy, M., Zaurin, R., & Catbas, F. N. (2007). *RELIABILITY OF A MOVABLE BRIDGE BY MEANS OF A FIELD CALIBRATED MODEL.* Washington: Transporter Research Board's 87th Meeting.

Thorogood, M. (2011). *Mechanical issues to consider when designing a moving bridge.* ICE Publishing.

Unknown. (s.d.). Obtido de http://members.tripod.com/joseph_berrigan/sitebuildercontent/sitebuilderpictures/pontoon.jpg

Unknown. (s.d.). *DrawBridgeAhead.com*. Obtido de http://www.drawbridgeahead.com/coreycauseway.html

Unknown. (s.d.). *ESDEP Course.* Obtido de Lecture 1B.6.1: Introduction to the Design of Steel and Composite Bridges: Part 1: http://fgg-web.fgg.uni-lj.si/~/pmoze/esdep/master/wg01b/l0610.htm

Unknown. (Unknown). *SECTION VI: MOVABLE BRIDGES*. Obtido de http://sha.md.gov/OPPEN/SECT_VI.pdf

Unknown. (s.d.). *www.resimhayattir.com*. Obtido de http://www.resimhayattir.com/r-mimari-resimler-186-millenium-bridge-6797.htm

Waddell, J. A. (1916). *Bridge Enginerring.* John Wilwy & Sons, Inc.

Yashinsky, M. (30 de April de 2010). *Bridge of the Week*. Obtido de Movable Bridges - Spokane Street Swing Bridge (2): http://www.bridgeofweek.com/2010/04/

A
APPENDICES

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output

Contents

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations			Remarks/Output
	2.0 Design parameters			
	Length of each cantilever span	34	m	
	Length of each back span	12	m	
	Overall width of the deck	23.4	m	
	Width of the deck walkways+cycleway	5	m	
	Length of the pier	22.5	m	
	Width of the pier	25	m	
	Height of the pier (front wall)	12.5	m	
	Height of the pier (back wall)	15.8	m	
	Deck at support	3.3	m	
	Deck at midspan	1.65	m	

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Summary: Inputs/Outputs	Remarks/Output

Inputs:

Back span:	12	m
Cantilevering span:	34	m
Total width:	22.4	m*
Cantilever width:	2	m
Curve radius	250	m
Number of boxes:	4	№
Width per box:	4.6	m
Max box Depth =	3.30	m
Min box Depth =	1.65	m
Top plate thickness =	50	mm
Web thickness =	35	mm
Bottom plate thickness =	50	mm
Additional flanges:		
Top depth =	0	mm
Top width =	0	mm
Bottom depth =	0	mm
Bottom width =	0	mm

*Excludes 2№ x 0.5m stringcourses

Loading

Assumed DL loading =	10	kN/m2
Calculated DL loading =	6.76	kN/m2

Steel properties

fy =	355	MPa
E =	210000	MPa

Transverse stiffener spacing

Assumed stiffener spacing	2	m

Outputs

Ratio check: 2.8 to 1 vs ideal 2.0 to 1

Check Bending stress on effective section:

Hogging	239.6	MPa
Sagging	37.0	MPa
UF	0.67	

Whole box width delivered to site? No. Need to split

Check Shear buckling:

Needs stiffeners:	TRUE	
Spacing check:	OK	*Stiffeners not designed*

Check reduced shear with moment:

$V_{b,Rd}$	OK
UF	0.13

Check flange induced buckling:	OK	2.40

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary loading calculations	Remarks/Output

3.0 Loading calculation and verification

3.1 Live Load

Estimated input

actual DL from geometry used

		L (m)	B (m)	M (kNm)	V (kN)
6.76 kN/m^2	10 kN/m^2	34	22.4	129472	7616

BS EN 1991-2:2003 CL 4.3.2

BS EN 1991-2:2003 CL 4.3.4

HA Tandem loads:	P (kN)		x (m)	M (kNm)	V (kN)	Remarks/Output
	300	at	34	10200	300	
	200	at	34	6800	200	
	100	at	34	3400	100	
	100	at	23	2300	100	gr5
	300	at	32.8	9840	300	
	200	at	32.8	6560	200	
	100	at	32.8	3280	100	
	100	at	21.8	2180	100	gr5

SV axles (SV80)	P (kN)		x (m)	M (kNm)	V (kN)	Remarks/Output
	150.8	at	34	5127	150.8	gr5
	150.8	at	32.8	4946	150.8	
	150.8	at	31.6	4765	150.8	
	150.8	at	30.4	4584	150.8	
	150.8	at	29.2	4403	150.8	
	150.8	at	28	4222	150.8	

Lane	UDL (kN/m2)	L (m)	B (m)	M (kNm)	V (kN)	Remarks/Output
Footways	5	34	6	17340	1020	
Lane 1	5.5	23	3	4364	380	gr5
Lane 1	5.5	34	3	9537	561	gr1a
Lane 2	5.5	34	3	9537	561	
Lane 3	5.5	34	3	9537	561	
Lane 4	5.5	34	3	9537	561	
Remaining	5.5	34	2.6	8265	486	

For central reservation

BS EN 1991-2-20003 CL4.2.3 note 4

Table 4.1 - Number and width of notional lanes

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_1 = 1$	3 m	w - 3 m
$5{,}4 \text{ m} \le w < 6$ m	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \le w$	$n_1 = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$

NOTE For example, for a carriageway width equal to 11m, $n_1 = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.

(3) For variable carriageway widths, the number of notional lanes should be defined in accordance with the principles used for Table 4.1.

NOTE For example, the number of notional lanes will be :
- 1 where $w < 5{,}4$ m
- 2 where $5{,}4 \le w < 9$ m
- 3 where $9 \text{ m} \le w < 12$ m, etc.

(4) Where the carriageway on a bridge deck is physically divided into two parts separated by a central reservation, then :

(a) each part, including all hard shoulders or strips, should be separately divided into notional lanes if the parts are separated by a permanent road restraint system ;

(b) the whole carriageway, central reservation included, should be divided into notional lanes if the parts are separated by a temporary road restraint system.

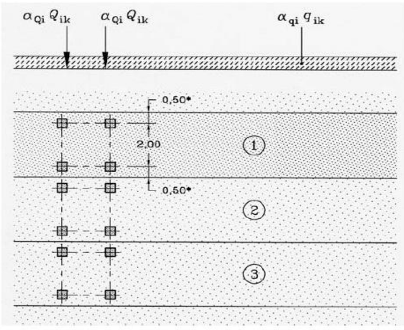
NOTE The rules given in 4.2.3(4) may be adjusted for the individual project, allowing for envisaged future modifications of the traffic lanes on the deck, *e.g.* for repair.

mouchel
building great relationships

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations			Remarks/Output
	Preliminary loading calculations			

Application of load model 1



Key
(1) Lane Nr. 1 : Q_{1k} = 300 kN ; q_{1k} = 9 kN/m^2
(2) Lane Nr. 2 : Q_{2k} = 200 kN ; q_{2k} = 2,5 kN/m^2
(3) Lane Nr. 3 : Q_{3k} = 100 kN ; q_{3k} = 2,5 kN/m^2
AC1 Tandem axle spacing = 1,2 m AC1
* For w_l = 3,00 m

3.2 Wind Load

Code Ref	Calculations	Value		Remarks/Output
BS EN 1991-1-4:2005+A1:2010 & NA	$v_{b,map}$ =	23 m/s		
	Site altitude =	5 m		
	Distance to shore =	0 km		Approx.
Assumed	structure height, z =	6 m	at service	
		23 m	when lifted	
NA.2.5	Altitude factor, c_{alt} =	1.005		

Procedure for determining the influence of altitude [BS EN 1991-1-4:2005, 4.2 (2)P Note 1]

The altitude factor c_{alt} should be determined from Equations NA.2a) or NA.2b).

(NA.2a)) $c_{alt} = 1 + 0{,}001 \cdot A$ for $z \leqslant 10$ m)

'(NA.2b)) $c_{alt} = 1 + 0{,}001 \cdot A \cdot (10/z)^{0.2}$ for $z > 10$ m

where

A is the altitude of the site in metres above mean sea level;

z is either z_s as defined in BS EN 1991-1-4:2005 Figure 6.1 or z_e the height of the part above ground as defined in BS EN 1991-1-4:2005 Figure 7.4.

Code Ref	Calculations	Value	Remarks/Output
NA 2.4	Fundamental value of basic wind velocity,	23.115 m/s	
NA.2.6	direction coefficient, c_{dir} =	1	Recommended conservative value
NA.2.7	season coefficient, c_{season} =	1	Recommended conservative value
4.2	Basic wind velocity, $v_b = v_{b0} \cdot C_{dir} \cdot C_{season}$ =	23.115 m/s	
NA.2.18	Air density, ρ =	1.226 kg/m3	
4.5-Expression (4.10)	Basic wind pressure, q_b =	327.528 N/m^2	
NA.2.11	Terrain category:	"Town"	
Fig NA.2	Orography not considered significant, therefore calculate peak velocity wind pressure as:		

Fig NA.2

$$q_p = c_e(z) \cdot c_{e,T} \cdot q_b$$

(obtain $c_e(z)$ from Figure NA.7;
obtain $c_{e,T}$ from Figure NA.8)

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output
	Preliminary loading calculations	

Calculate $q_p(z)$ for relevent structural elements at height above ground, (z)

Basic wind pressure, q_b = 327.528 Pa (Calculated on previous sheet)
c_{et} is taken conservatively as 1.0 as distance inside 'town' is not clear
c_e is based on a distance to shore of 0km

Peak velocity wind pressure, $q_p = c_e(z).c_{eT}.qb$

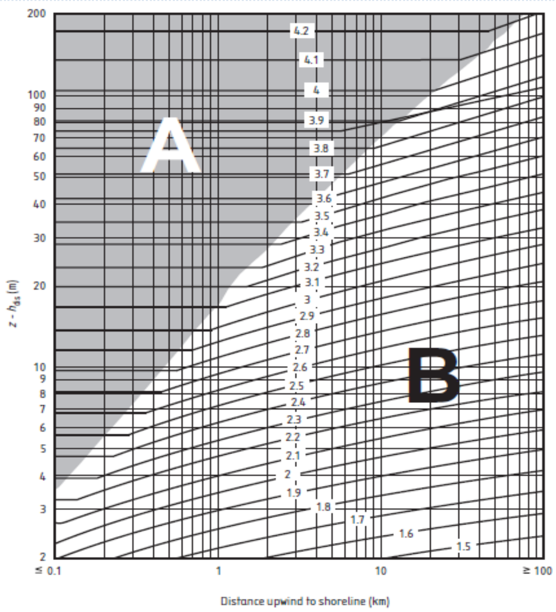
Structural element	Av. Height, z (m)	$c_{e(z)}$	c_{et}	$q_p(z)$ (kPa)
Deck	0	3.2	1.0	1.0

Wind on parapet and deck beams is considered seperately

Take d_{tot} as 2.5m for deck beam only — Average depth

8.3.1 Fig 8.3 — Take b as full width of 23.4m

Code Ref		Value
	b/d =	9.36
Fig 8.3	$c_{fx,0}$ =	1.30
Fig NA.7	c_e =	3.20
Table NA.7	C =	4.00



NOTE 1 The height z is the height at which q_p is sought using Equations NA.3a) or NA.3b).
NOTE 2 Zones A and B are indicated for use in Table NA.3.

	Value	Unit
$A_{ref,x}$ =	2.50	m^2/m
v_b =	23.12	m/s
ρ_{air} =	1.23	kg/m^3
Force on beam only =	3.28	kN/m

For parapets:

	Value	Unit	Remarks
assume solidity ratio, ϕ =	0.80	m^2/m^2	* TBC *
$c_{fx,0}$ =	1.30		
q_p =	1.05	kPa	
parapet height =	1.40	m	
$A_{ref,x}$ =	1.12	m^2/m	
Force on one parapet only	1.53	kN/m	
Total force FX =	6.33	kN/m	

	Value	Unit
Beams FY	0.82	
Parapet FY	3.05	
Total force FY =	3.87	kN/m
	1.17	kN/m^2

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary loading calculations	Remarks/Output

Summary:

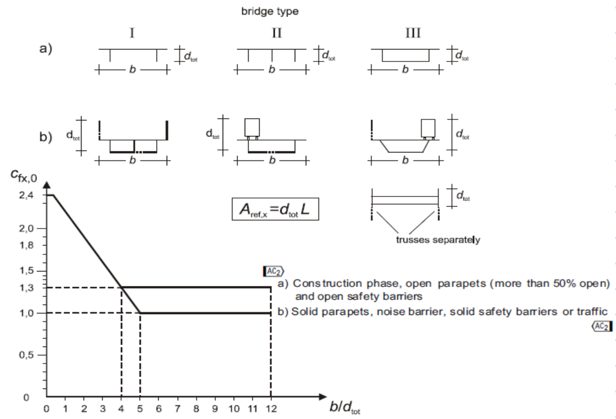
	FX	FY
Option 1 - Separate	6.3	3.9

FX= wind transversely applied to the girder
FY= longitudinally applied along the footway

Vertical wind loading on footway:

A- Consider when the bridge is lifted (acting perpendicular on the footway or soffit of the bridge)

Code Ref			
EN 8.3.3 Fig 8.3	$c_{fz} = c_{fx}$	2.1	
	c_e =	3.2	
8.3.2	C =	6.72	



A_{refz} =	23.4	m^2 per m run			
ρ =	1.226	kg/m^3			
v_b =	23.115	m/s			
F =	51.5031	kN/m			
Moment =	28754.4	kN.m	midas	28880	kN.m
Shear =	1751.11	kN	value =	1748	kN

B- Consider when the bridge is in service

c_{fz}	0.9		recommended
c_e =	3.2		
C =	2.88		

A_{refz} =	23.4	m^2 per m run			
ρ =	1.17299	kg/m^3			
v_b =	23.115	m/s			
F =	21.1183	kN/m			
Moment =	12206.4	kN.m	midas	13068	kN.m
Shear =	718.023	kN	value =	769	kN

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary loading calculations	Remarks/Output

3.3 Snow load

For site in area 3 at sea level
Assume 10m altitude for conservatism.

NA.4.1.1 *Snow loading on bridges*

Snow loads should be considered in accordance with local conditions. For those local conditions prevailing in the United Kingdom, this loading may generally be ignored. However, there are circumstances, e.g. for opening bridges, covered bridges including roofed footbridges, or where stability due to permanent loads is critical, when snow loading should be taken into account in the design.

On ordinary bridges the accumulation of any material quantity of snow will effectively reduce the traffic loads, probably by amounts greater than the snow loads, but certainly to such an extent that the combined mass of snow and traffic loading will not exceed the nominal live load. On opening bridges, however, snow load can be significant. These bridges are likely to be designed to move under permanent loads only and the added weight could have serious adverse effects on machinery and moving parts. On swing bridges too, where snow might have been cleared from one wing of the rotating structure but not the other, the stability against overturning could be impaired.

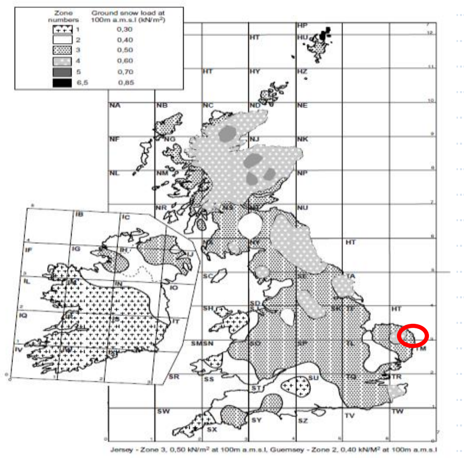


Figure NA.1 — Characteristic ground snow load map

Snow load with live load is not a significant combination for bridges, but snow plus dead load is significant for movable bridges.

UK NA does not contain guidance on γ for snow loading as it can largly be ignored for UK bridges. However, it needs to be checked for a movable bridge.

Consider factor from main eurocode 1990 as 1.5 'For all other variable actions'

$$s_k = [0{,}15 + (0{,}1Z + 0{,}05)] + \left(\frac{A-100}{525}\right) \quad \text{(NA.1)}$$

Altitude =	10	m
Zone =	3	
Snow load at site =	0.32857	kN/m^2
γsnow =	1.5	
total snow load =	353.674	kN
centroid =	23	m
lever to pivot =	11	m
moment =	3890.42	kNm at pivot point

factored

total snow load =	530.511	kN
moment =	5835.63	kNm at pivot point

3.4 Dead load

Superstructure dead load was calculated based on the total area of the deck depending on the deck option, below is an example for three trapezoidal box

Current self weight estimate assumes: **10.0** kN/m^2

Actual deadweight is approximately the average of:		165.0	kN/m	3.3m depth
		90.0	kN/m	1.65m depth
	=	127.5	kN/m	

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary loading calculations	Remarks/Output

Over total width = 22.4 m

Assumed

Weight per unit area = 5.7 kN/m^2
plus surfacing = (based on 10mm @18kN/m^3) 0.2 kN/m^2
plus parapets =(based on 2 @ 1kN/m) 0.1 kN/m^2
Add nominal 0.2 ers and connections: 1.1 kN/m^2
7.1 kN/m^2

BS EN 1991-2:2003 CL 4.4.1.2

3.5 Longitudnal load

The longitudnal live load for SV80 = 900 kN

3.6 Load combinations

Consider worst of Gr1a and Gr5 loading:

Moments	**gr1a**		**gr5**			Code Ref	Remarks/Output
Factor	γ	γQ + γq	γ	ϕ1	γQ + γq		
LM1	1.35	116766	1.35	0.75	80110		*113535*
LM3	0	0	1.35	1	37866		96%
Pedestrians	1.35	14045.4	0	1	0		
DL	1.2	155366	1.2	1	155366		
WIND*	*1.7 x 0.2*	4144.61	*0.7*	0.2	4144.6	See below	
M Sum		290322			277487		

Take maximum M = 291000 kNm hogging

Shears			gr5			Code Ref	Remarks/Output
Factor	gr1a		γ	ϕ1	γQ + γq		
LM1	1.35	5306	1.35	0.75	3796		
LM3	0	0	1.35	1	1221		5017
Pedestrians	1.35	826	0	1	0		
DL	1.2	9139	1.2	1	9139.2		
WIND*	*1.7 x 0.2*	243.8	*0.7*	0.2	243.8	See below	
M Sum		15515			14400		

Take maximum V = 16000 kN

Maximum sagging case - Bridge lifted. Wind as leading variable.

Deadload 6.76 kN/m2
Length of cantilever 34 m — 0.16888

Lever for deadload:
Lift angle = 83 °
horizontal lever = 2.07 m to centre
moment SW = 8916 kNm 4303.34

Factor on favourable DL 0.95

Factored restoring moment from deadload = 8469.79 kNm 4088.17

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary loading calculations	Remarks/Output

*From separate wind loading calculation:

γ_q on wind =	1.7			
ψ_1 on wind as leading variable action =	1.0			
ψ_1 on wind as non leading variable action =	0.2			

<u>Hogging result (bridge in service posiiton)</u>

UDL wind =	21.1	kN/m	Factored:	
Shear due to wind =	717.06	kN	243.8	kN
Moment due to wind =	12190	kNm	4144.6	kNm

<u>Sagging result (bridge in lifted 83 position)</u>

Lifting angle =	83	°		
UDL wind =	51.4	kN/m	Factored:	
Shear due to wind =	1748.42	kN	2972.3	kN
Moment due to wind (calculated) =	29502	kNm	50152.6	kNm

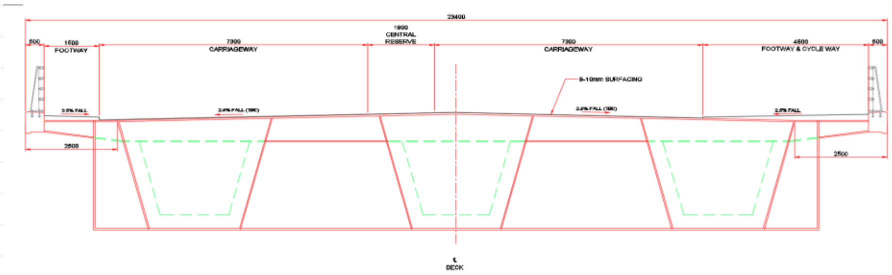
Net sagging effect in lifted position =	41682.8	kNm	7060.5	kN
Deflection =	51	mm from Midas		
Deflection amounts to an additional angle of	0.34	degrees		

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary Section Capacity Check: 3No. trapezoidal box (hogging)	Remarks/Output

4.0 Deck design

4.1 At support (hogging)



Hogging moment

Checking maximum cantilever moment H = 3300 mm

Carried from previous inputs

Total width	22400	mm	№ boxes	3	№
cantilever width	2500	mm	box width	4200	m
fy =	355	MPa			

	b	d	№	b_{sum}	A	c	I
Edge cantilevers	2500	50	2	5000	250000	3275	5.2E+11
Half top plate	2100	50	6	12600	630000	3275	1.3E+12
Top flanges	0	0	0	0	0	3250	0
Web	35	3200	6	210	672000	1650	6E+11
bottom flanges	0	0	0	0	0	50	0
Half btm plate	2100	50	6	12600	630000	25	2.1E+12
					2182000		4.5E+12

y_c = 1836.18 mm

Iyy = 4.5E+12 mm4
ztop = 3.1E+09 mm3
zbtm = 2.4E+09 mm3

M = 291000 kNm
σT = 95.0 MPa
σB = 119.1 mpa

Therefore appears to be ok assuming whole section is effective and suitably stiffened

Discounting a portion of plate for shear lag and class 4 effective section properties:
Reduction factors are calculated further below for effective width and shear lag

Indicates effective dimensions

H = 3300

	b	d	№	bsum	A	c	I
Edge cantilevers	2473	50	2	4945.94167	247297.08	3275	2.5E+11
Half Top plate	2083	50	6	12495.01067	624750.53	3275	6.4E+11
Top flanges	0	0	0	0	0	3250	0
Web	35	985	6	210	206892	2757.4	6.7E+10
Neglected web		0					
Web	35	2215	6	210	465108	1157.4	7.6E+11
bottom flanges	0	0	0	0	0	50	0
Half btm plate	697	50	6	4182.145247	209107.26	25	1E+12

yc = 2264.48 mm
Af = 1753155 mm²
Iyy = 2.8E+12 mm4
ztop = 2.7E+09 mm3
zbtm = 1.2E+09 mm3

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output
	Preliminary Section Capacity Check: 3No. trapezoidal box (hogging)	

M = 291000 kNm
σT = 108.9 MPa
σB = 238.2 MPa

τw = 23.81 MPa

Reduced effective section should work providing that bracing and stiffeners are adequate

Ifk = 1.9E+12
Mfk = 664,887 kNm
gammaM0 = 1
MfRd = 664,887 kNm

First check shear lag: **(Effective^{S})**

BSEN 1993-5-2006 Table 3.1 & Fig 3.2

For cantilever, Le = 2L = 34 m *(Le = L, conservative for shear lag)*
Area of stiffeners, Asl = 0 mm²
$k = a_0 b_0 / L_e$

Consider:	t	b0	a0	*k*	β sag	β hog
Edge cantilever	50	2482.5	1	0.073	0.967	0.989
Half top plate	50	2082.5	1	0.061	0.977	0.992
top flange	0	0	0	0.000	1.000	1.000
bottom flange	0	0	0	0.000	1.000	1.000
bottom plate	50	2082.5	1	0.061	0.977	0.992

Check effective area due to buckling (Effective^{P})

BS EN 1993-5-2006 4.4

	b	d	№	A	c	σ	
Edge cantilever	2473	50	2	247297.0835	3275	355.0	
Half top plate	2083	50	6	624750.5336	3275	355.0	
top flange	0	0	0	0	3250	346.2	
web in tension	35	985.2	6	206892	2757.4	173.2	
web in compression	35	2214.8	6	465108	1157.4	-175.5	
bottom flange	0	0	0	0	50	-351.0	
bottom plate	697	50	6	209107.2624	25	-355.0	compressive

Manually update!

Will need update if effective web requires reduction

centroid = 2264.48 mm
therefore web in compression = 2214.48 mm

Assume yield strain is reached at compressive plate's centre:

stress at bottom flange cl = 25 mm -355 MPa
stress at centroid = 0 d
stress at top of web, 3250 mm 156.22366
stress at bottom of web = 50 mm -351.03703

$\psi = \sigma_2/\sigma_1$ = -2.2

BS EN 1993-5-2006 4.4 Tables 4.1&4.2

ψ = 1 FALSE
1 > ψ > 0 FALSE
ψ = 0 FALSE
0 > ψ > -1 FALSE
-1 > ψ > -3 TRUE Therefore take kσ = 63.0

k_σ = 63.0

BS EN 1993-5-2006 4.4.2

Slenderness of the web: λp = 0.501

Limit check: λ < 0.5 + √ (0.085-0.055ψ) = 0.96 TRUE
Therefore use ρ = 1.00
beff = 2214

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output
	Preliminary Section Capacity Check: 3No. trapezoidal box (hogging)	

Repeat for flanges/bottom plate:

for flanges, ψ = 1.0 *conservative*

$\psi = 1$	TRUE	Therefore take $k\sigma$ =	0.43
$1 > \psi > 0$	FALSE		
$\psi = 0$	FALSE		
$0 > \psi > -1$	FALSE		
$\psi = -1$	FALSE		

for flanges, $k\sigma$ = 0.43

Slenderness of the bottom plate: λp = 2.761

Limit check: $\lambda < 0.748$ 0.75 FALSE

Therefore use ρ = 0.34

Effective plate width = 697.0 mm

Slenderness of the bottom flange λp = 0.000 zero if no flange

Limit check: $\lambda < 0.748$ 0.75 TRUE

Therefore use ρ = 1.00

Effective plate width = 0 mm

BS EN 1993-5-2006 5.1

Total applied shear: 16000 kN

Total section area = 2182000 mm²

V/A = 7.3 N/mm²

Web area only = 672000 mm²

23.8 N/mm²

<u>However, need to check succeptability to shear buckling:</u>

hw =	3200	mm
tw =	35	mm
hw/tw =	91.428571	
ϵ =	0.81	
η =	1.0	
Transverse stiffener spacing, a =	2000	mm
Shear buckling coefficient, $k\tau$ =	18	

Unstiffened shear buckling:	$hw/tw > 72\epsilon/\eta$	TRUE	∴ Needs stiffeners	
Stiffened shear buckling:	$hw/tw > 31\epsilon/\eta\sqrt{k_\tau}$	FALSE	Assumed spacing ok	OK

<u>Check reduced shear capacity assuming no stiffeners and rigid end post:</u>

Modified slenderness λw (for EC3-1-5 Table 5.2)

fyw= fyf=	355	MPa
τcr =	401.639	MPa
$k\tau$ =	17.6704	(ignoring L stiffener term - hence assuming no L stiffeners present)
σE =	22.7295	
a/hw =	0.625	
$\gamma M1$ =	1.1	
λw =	0.715	
χ =	1.00	(assuming a rigid end post is used)

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary Section Capacity Check: 3No. trapezoidal box (hogging)	Remarks/Output

BS EN 1993-5-2006 5.2

For unstiffened or stiffened webs the design resistance for shear should be taken as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta\ f_{yw}\ h_w\ t}{\sqrt{3}\ \gamma_{...}}$$

$$V_{bw,Rd} = \frac{\chi_w\ f_{yw}\ h_w\ t}{\sqrt{3}\ \gamma_{M1}}$$ 20869 kN per web

Contribution from flanges:

tf =	50	mm
bf =2 x15ϵ.tf =	1220	mm per web
a =	2000	mm
c =	527.2416243	mm per web
Mf,Rd =	664,887	kNm
Vbf =	1,510	kN per web
Total web+flanges limited by:	20869	kN per web
№ webs =	6	№
Total effective resistance (Entire section considered) =	125,212	kN
Total applied shear =	16,000	kN

OK!

Remarks/Output: OK

Check Flange Induced Buckling

BS EN 1993-5-2006 8.1

$$\frac{h_w}{t_w} \leq k\ \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$ TRUE — *Therefore ok ignoring radius*

hw =	3200	mm
t w=	35	mm
k =	0.55	for elastic moment utilised
E =	210000	MPa
fyf =	355	MPa
Aw =	112000	mm²
Afc = (used bottom plate. No flange)	69702.421	mm²

Remarks/Output: *worse case if plastic(0.4)*

Must assume the bottom flange is curved. Hence:

r = 250.000 m

$$\frac{h_w}{t_w} \leq \frac{k\ \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}}{\sqrt{1 + \frac{h_w E}{3\ r\ f_{yf}}}}$$ TRUE — Therefore ok considering radius

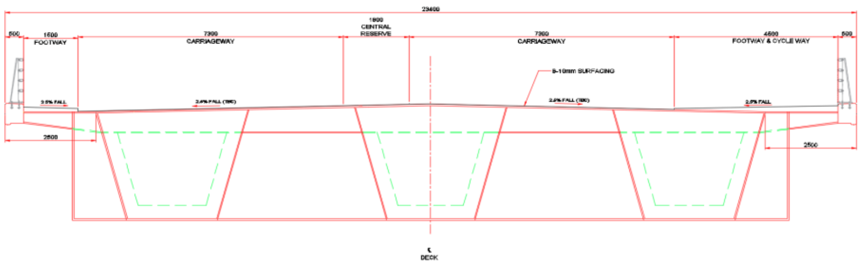
r is the radius of curvature of the compression flange.

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary Section Capacity Check: 3No. trapezoidal box (sagging)	Remarks/Output

4.0 Deck design

4.2 At support (sagging)



Hogging moment

Checking maximum cantilever moment H = 3300 mm — Carried from previous inputs

Total width 22400 mm — № boxes 3 №
cantilever width 2500 mm — box width 4200 m
fy = 355 MPa

	b	d	№	b_{sum}	A	c	I
Edge cantilevers	2500	50	2	5000	250000	3275	5.2E+11
Half top plate	2100	50	6	12600	630000	3275	1.3E+12
Top flanges	0	0	0	0	0	3250	0
Web	35	3200	6	210	672000	1650	6E+11
bottom flanges	0	0	0	0	0	50	0
Half btm plate	2100	50	6	12600	630000	25	2.1E+12
					2182000		4.5E+12

y_c = 1836.18 mm

Iyy = 4.5E+12 mm4
ztop = 3.1E+09 mm3
zbtm = 2.4E+09 mm3

M = 45000 kNm
σT = 14.7 MPa
σB = 18.4 mpa

Therefore appears to be ok assuming whole section is effective and suitably stiffened

Discounting a portion of plate for shear lag and class 4 effective section properties:
Reduction factors are calculated further below for effective width and shear lag

Indicates effective dimensions — H = 3300

	b	d	№	bsum	A	c	I
Edge cantilevers	2473	50	2	4945.9417	247297.08	3275	2.5E+11
Half Top plate	2083	50	6	12495.011	624750.53	3275	6.4E+11
Top flanges	0	0	0	0	0	3250	0
Web	35	985	6	210	206892	2757.4	6.7E+10
Neglected web		0					
Web	35	2215	6	210	465108	1157.4	7.6E+11
bottom flanges	0	0	0	0	0	50	0
Half btm plate	697	50	6	4182.1452	209107.26	25	1E+12

yc = 2264.48 mm
Af = 1753155 mm²
Iyy = 2.8E+12 mm4
ztop = 2.7E+09 mm3
zbtm = 1.2E+09 mm3

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output
	Preliminary Section Capacity Check: 3No. trapezoidal box (sagging)	

M = 45000 kNm
σT = 16.8 MPa
σB = 36.8 MPa

τw = 23.81 MPa

Reduced effective section should work providing that bracing and stiffeners are adequate

Ifk = 1.94E+12
Mfk = 304,045 kNm
gammaM0 = 1
MfRd = 304,045 kNm

First check shear lag: **(Effective[S])**

BSEN 1993-5-2006 Table 3.1 & Fig 3.2

For cantilever, Le = 2L = 34 m *(Le = L, conservative for shear lag)*
Area of stiffeners, Asl = 0 mm²
$k = a_0 b_0 / L_e$

Consider:	t	b0	a0	*k*	β sag	β hog
Edge cantilever	50	2482.5	1	0.073	0.967	0.989
Half top plate	50	2082.5	1	0.061	0.977	0.992
top flange	0	0	0	0.000	1.000	1.000
bottom flange	0	0	0	0.000	1.000	1.000
bottom plate	50	2082.5	1	0.061	0.977	0.992

Check effective area due to buckling (Effective[P])

BS EN 1993-5-2006 4.4

Manually update!

	b	d	№	A	c	σ	
Edge cantilever	2473	50	2	247297.08	3275	355.0	
Half top plate	2083	50	6	624750.53	3275	355.0	
top flange	0	0	0	0	3250	346.2	
web in tension	35	985.2	6	206892	2757.4	173.2	
web in compression	35	2214.8	6	465108	1157.4	-175.5	
bottom flange	0	0	0	0	50	-351.0	
bottom plate	697	50	6	209107.26	25	-355.0	compressive

Will need update if effective web requires reduction

centroid = 2264.48 mm
therefore web in compression = 2214.48 mm

Assume yield strain is reached at compressive plate's centre:

stress at bottom flange cl = 25 mm -355 MPa
stress at centroid = 0 d
stress at top of web, 3250 mm 156.22366
stress at bottom of web = 50 mm -351.03703

$\psi = \sigma_2/\sigma_1 =$ -2.2

BS EN 1993-5-2006 4.4 Tables 4.1&4.2

ψ = 1 FALSE
1 > ψ > 0 FALSE
ψ = 0 FALSE
0 > ψ > -1 FALSE
-1 > ψ > -3 TRUE Therefore take kσ = 63.0

$k_\sigma =$ 63.0

BS EN 1993-5-2006 4.4.2

Slenderness of the web: λp = 0.501

Limit check: λ < 0.5 + √ (0.085-0.055ψ) = 0.96 TRUE
Therefore use ρ = 1.00
beff = 2214

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Remarks/Output
	Preliminary Section Capacity Check: 3No. trapezoidal box (sagging)	

Repeat for flanges/bottom plate:
for flanges, ψ = 1.0 *conservative*

ψ = 1	TRUE	Therefore take kσ =	0.43
1 > ψ > 0	FALSE		
ψ = 0	FALSE		
0 > ψ > -1	FALSE		
ψ = -1	FALSE		

for flanges, kσ = 0.43

Slenderness of the bottom plate: λp = 2.761

Limit check: λ < 0.748 — 0.75 — FALSE
Therefore use ρ = 0.34

Effective plate width = 697.0 mm

Slenderness of the bottom flange λp = 0.000 zero if no flange

Limit check: λ < 0.748 — 0.75 — TRUE
Therefore use ρ = 1.00

Effective plate width = 0 mm

BS EN 1993-5-2006 5.1

Total applied shear: 16000 kN

Total section area = 2182000 mm²
V/A = 7.3 N/mm²

Web area only = 672000 mm²
23.8 N/mm²

However, need to check succeptability to shear buckling:

hw =	3200	mm
tw =	35	mm
hw/tw =	91.428571	
ϵ =	0.81	
η =	1.0	
Transverse stiffener spacing, a =	2000	mm
Shear buckling coefficient, kτ =	18	

Unstiffened shear buckling:	hw/tw > 72ε/η	TRUE	∴ Needs stiffeners	
Stiffened shear buckling:	hw/tw > 31ε/η√k_τ	FALSE	Assumed spacing ok	OK

Check reduced shear capacity assuming no stiffeners and rigid end post:

Modified slenderness λw (for EC3-1-5 Table 5.2)

fyw= fyf=	355	MPa
τcr =	401.639	MPa
kτ =	17.6704	(ignoring L stiffener term - hence assuming no L stiffeners present)
σE =	22.7295	
a/hw =	0.625	
γM1 =	1.1	
λw =	0.715	
χ =	1.00	(assuming a rigid end post is used)

BS EN 1993-5-2006 5.2

For unstiffened or stiffened webs the design resistance for shear should be taken as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \le \frac{\eta\, f_{yw}\, h_w\, t}{\sqrt{3}\, \gamma_{M1}}$$

$$V_{bw,Rd} = \frac{\chi_w\, f_{yw}\, h_w\, t}{\sqrt{3}\, \gamma_{M1}}$$

20869 kN per web

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Preliminary Section Capacity Check: 3No. trapezoidal box (sagging)	Remarks/Output

<u>Contribution from flanges:</u>

tf =	50	mm
bf =2 x15ε.tf =	1220	mm per web
a =	2000	mm
c =	527.24162	mm per web
Mf,Rd =	304,045	kNm
Vbf =	1,827	kN per web
Total web+flanges limited by:	20869	kN per web
№ webs =	6	№
Total effective resistance (Entire section considered) =	125,212	kN
Total applied shear =	16,000	kN

OK!

Remarks/Output: OK

Check Flange Induced Buckling

BS EN 1993-5-2006 8.1

$$\frac{h_w}{t_w} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

TRUE — *Therefore ok ignoring radius*

hw =	3200	mm
t w=	35	mm
k =	0.55	for elastic moment utilised
E =	210000	MPa
fyf =	355	MPa
Aw =	112000	mm²
Afc = (used bottom plate. No flange)	69702.421	mm²

worse case if plastic(0.4)

Must assume the bottom flange is curved. Hence:

r = 250.000 m

$$\frac{h_w}{t_w} \leq \frac{k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}}{\sqrt{1 + \frac{h_w E}{3\, r\, f_{yf}}}}$$

TRUE Therefore ok considering radius

r is the radius of curvature of the compression flange.

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Edge cantilever	Remarks/Output

4.3 Edge cantilever

Consider moment, shear and axial forces on the cantilever element at:

Section 1	the junction with the main longitudinal elements (thick end)
Section 2	the junction with the edge beam (thin end)

Assume impact loading is carried fully by a single cantilever

Assume cantilever spacing =	3	m

Section properties:

Max depth	700	mm
min depth	300	mm
web thickness	25	mm
top plate width	500	mm
top plate depth	50	mm
bottom flange width	350	mm
bottom flange depth	25	mm

Note: The PNA calc assumes that the PNA is in the top flange. Check if this is valid. If not, will need to change the PNA calc

Valid? Yes, Valid

Loading calculation:

Cantilever length =	2	m
impact force =	500	kN
impact height =	1.25	m
wheel load =	600	kN
wheel position on cantilever =	1.7	m

Self weight loads

parapet	1	kN/m	3.0	kN per cantilever	2.25
edge beam	1.66	kN/m	5.0	kN per cantilever	2.25
top plate	7.7	kN/m	23.1	kN per cantilever	1.0
web	-	kN/m	1.64	kN per cantilever	1.0
flange	-	kN/m	1.35	kN per cantilever	1.0

SW moment at section 1 (at deck) =	44.1	kNm
SW moment at section 2 (at parapet) =	2.0	kNm
SW shear at section 1 (at deck) =	34.1	kN
SW shear at section 2 (at parapet) =	8.0	kN

load effects on section 1 At deck

axial =	500	kN
moment =	1689.0604	kNm
shear =	634.1	kN

load effects on section 2 At parapet

moment =	44.06035	kNm
moment =	626.9974	kNm
shear =	607.9896	kN

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Edge cantilever	Remarks/Output

Section properties

Section 1

b (mm)	d (mm)	A (mm^2)	y (mm)	Iyy (mm^4)	Izz (mm^3)
500	50	25000	675	1.262E+09	520833333
25	625	15625	337.5	709171337	813802.08
350	25	8750	12.5	1.681E+09	89322917

Av = 15625
Vplrd = 3202 kN
20% ok

Section 2

b (mm)	d (mm)	A (mm^2)	y (mm)	Iyy (mm^4)	Izz (mm^3)
500	50	25000	275	157215490	520833333
25	225	5625	137.5	43660316	292968.75
350	25	8750	12.5	298384797	89322917

Av = 5625
Vplrd = 1153 kN
53% ok

As UF in shear for section 2 (parapet) exceeds 50%, need to consider moment/shear interaction:

$\rho = (2 \cdot V_{ED}/V_{pl,Rd} - 1)^2 =$ 0.003
Reduction factor $(1-\rho)$ = 0.997
Apply this to section 2 check below

Check section classification:

flange: c = 162.5 mm
t = 25 mm
c/t = 6.5 Class 1 flange? TRUE

web: c = 625 mm
t = 25 mm
c/t = 25.0 mm Class 1 web? TRUE

Section 1 c/t ratio is critical. Therefore will be class 1 along full length

Section is class 1

Section 1 Mpl:

	b (mm)	d (mm)	A (mm^2)	y (mm)
Top flange T	500	49.375	24687.5	24.6875
Top flange C	500	0.625	312.5	0.3125
Web	25	625	15625	312.813
Bottom flnage	350	25	8750	638.125

AT 24687.5
AC 24687.5

PNA 650.6 mm
x 0.6 mm
ZP = 1.1E+07 mm3

MP = 3933.7 kNm
MP reduced (Shear) = 3921.9 kNm
Required moment = 1689.1 kNm 43% Utilised Ok

Moment check using only nominal 500mm of top plate is acceptable. LTB reduction will apply, but will not be significant over 2m length, and additional plate width or additional top flange could be used to increase capacity if necessary

	b (mm)	d (mm)
Depth	-	300
Top falnge	500	50
Web	25	225
Bottom flange	350	25

Repeat for section 2

	b (mm)	d (mm)	A (mm^2)	y (mm)
TF T	500	39.375	19687.5	19.6875
TF C	500	10.625	5312.5	5.3125
W	25	225	5625	117.813
BF	350	25	8750	248.125

AT 19687.5
AC 19687.5

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Edge cantilever	Remarks/Output

PNA 260.6 mm
x 10.6 mm

ZP = 3249609 mm3

MP = 1153.61 kNm
Required moment = 630 kNm 55% Utilised

Check mid point of taper:

	b	d	A	
TF T	500	43.3375	21668.8	21.6688
TF C	500	6.6625	3331.25	3.33125
W	25	112.5	2812.5	59.5813
BF	350	25	8750	131.663

AT 21668.8
AC 14893.8

PNA 144.163 mm
x 6.6625 mm

ZP = 1800251 mm3

MP = 639.09 kNm

Required moment = 630 kNm 99% Utilised

Section cannot buckle at this point. Hence connection looks ok

x	moment capacity	moment required	
0	3922	1689	43%
1	2538	1158	46%
2	1154	627	54%

Section is nearest capacity at midpoint, but it seems that 300 and 700mm are workable as dimensions.

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Counterweight	Remarks/Output

4.4 Kentledge

Counterweight will be used to minimise the deal load, a preliminary calculation to check feasibility of the counterweight is shown below. However, KGAL consulting suppose to inform us regarding this calculation;

Approx moment from self weight of the box section:		
Area of min. section	1464724	mm²
Area of max section	2182000	mm²
Area of backspan 1 (solid section)	2182000	mm²
Area of backspan 2 (tapered section)	1464724	mm²
Main section 1		
Length =	4	m
Area	2182000	mm²
Force	672.06	kN
Lever	2.00	m
Taper (constant component)		
Length	30	m
Area	1464724	mm²
Force	3383.51	kN
Lever	19.00	m
Taper (varying component)		
Length =	30	m
Area	358638	mm² average
Force	828.45	kN
Lever	14.00	m
Backspan 1		
Length =	4	m
Area	2182000	mm²
Force	672.06	kN
Lever	2.00	m
Backspan 2		
Length =	8	m
Area	1464724	mm²
Force	902.27	kN
Lever	8.00	m

Surfacing	138	kN	at	23	m
Parapets	68	kN	at	23	m
Edge beams	214	kN	at	23	m

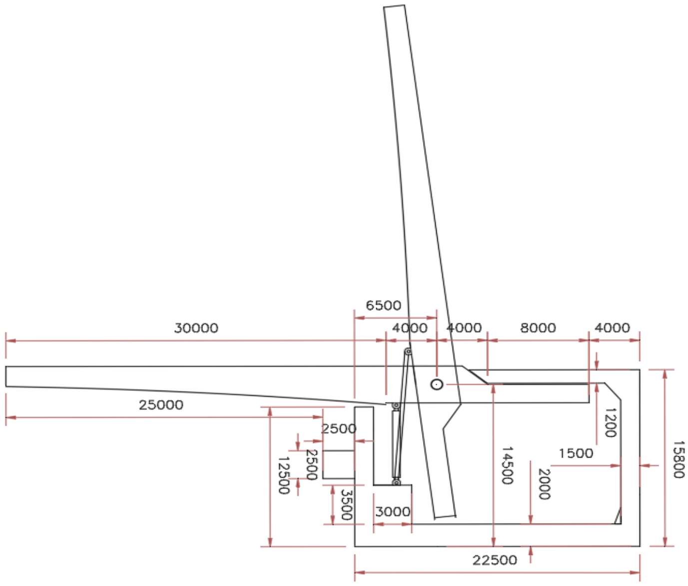
Total weight steel x1.2 plus SDL =	8169.3	kN
Centroid of total section (from pivot point) =	11.39	m
Centroid of total section (from backspan end) =	11.60	m
Counterweight must balance:	94764	kNm
assume kentledge is in end of BS2:		
L kentledge =	3	m
therefore lever =	12.00	
kentledge force =	7897.0357	kN
kentledge density =	113	kN/m3 (assume lead)
kentledge volume =	69.9	m3

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

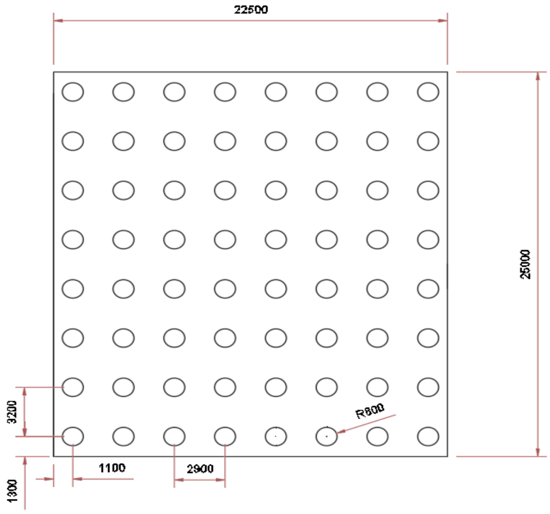
Code Ref	Calculations Pier design	Remarks/Output

5.0 Pier and Foundation

Pier and foundation arrangements are shown below



Pier elevation



Foundation arrangement

Design of the pier's back wall is very critical due to earth pressure, hence design of this wall was considered to be significant, see below;

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations	Pier design				Remarks/Output
	5.1 Pier wall design					
	All calculations are considered per metre width - ignoring any benefit from the leaf walls					
TBC From drawing	Total wall height	15.8	m			
	Wall thickness at haunch	2	m			
	Wall thickness above haunch	1.5	m			
	Base slab depth	2	m			
	Haunch height	1	m			
	Wall depth inc. haunch	13.8	m			
	Wall depth to haunch	12.8	m			
	overhang length	5	m			
	overhang thickness	0.4	m			
	ρ_c	25	kN/m^3			
	$\gamma_{q,c}$	1.35				
	Assumed soil properties:					
PD6694 for 6N Fill	K_a =	0.33				
	ρ_s =	19	kN/m^3			
	h(in haunch) =	13.8	m			
	h(wall only)	12.8	m			
	γ =	1.35				
	Moment due to EP =	3708	kNm / m	2959	kNm / m	
	Shear due to EP =	806	kN / m	693	kN / m	
	Moment due to overhang =	168.8	kNm / m	168.8	kNm / m	
	Surcharge =					
PD6694	surcharge =	6.6	kN/m2			
EC0	factor =	1.35				
	shear =	91.08	kN / m	84.48	kN / m	
	moment =	628	kNm / m	541	kNm / m	
	Moment due to long. Traffic	230.0	kNm / m	213.3	kNm / m	
	Shear due to long. Traffic	16.7	kN / m			
EC0	factor =	1.35				
	factored moment =	310.5	kNm / m	288	kNm / m	
	factored shear =	22.5		22.5		
	Total factored moment =	4815	kNm / m	3956	kNm / m	
	Total factored shear	920	kN / m	800	kN / m	

Moment check		
UF =	0.55	haunch
UF =	0.62	wall
Shear check (No links)		
UF =	1.01	wall

Code Ref	Moment Capacity at haunch			Moment Capacity at wall (top of haunch)		
Assumed C40/50	f_{ck} =	40	MPa			
EC2, γ_c = 1.5	f_{cd} =	26.7	MPa			
	total depth, h =	2000	mm	total depth, h =	1500	mm
	bar diameter 1 =	32	mm	bar diameter 1 =	32	mm
	bar spacing 1 =	150	mm	bar spacing 1 =	150	mm
	cover 1 =	55	mm	cover 1 =	55	mm
	bar diameter 2 =	32	mm	bar diameter 2 =	32	mm
	bar spacing 2 =	150	mm	bar spacing 2 =	150	mm
	cover 2 =	97	mm	cover 2 =	97	mm
	A_s =	10723	mm²	A_s =	10723	mm²
	cover to centroid of steel	76	mm	cover to centroid of steel	76	mm
	eff depth =	1908	mm	eff depth =	1408	mm
	F_s =	4662	kN	F_s =	4662	kN
	x =	108	mm	x =	108	mm
	Z =	1865	mm	Z =	1365	mm
	M =	8695	kNm	M =	6363	kNm

Project	Part of structure/scheme and status	Date
Great Yarmouth Third River Crossing	General	Jul-17

Code Ref	Calculations — Pier design	Remarks/Output

Code Ref				
	Design shear =	800.392	kN / m	Consider across wall only. Shear enhancement at haunch.
Taking 1m	width, b =	1000	mm	
	Eff depth, d =	1408	mm	
	ρ =	0.762%		
UK NA to EC2	k =	1.38		
	V_{Rdc} =	726.6		
UK NA to EC2	vmin =	0.358		
	$V_{Rdc, MIN}$ =	503.558	kN / m	
UK NA to EC2	$C_{Rd,c}$ =	0.12		
	$V_{Rd,c}$ =	726.6	kN / m (Ignoring axial force)	

If axial force from self weight of concrete is considered:

k_1 =	0.15	
F_Z =	432.538	kN / m
σ_c =	0.3072	MPa
V_{Rd} =	791.5	kN / m

Therefore shear links would be required

Shear capacity including shear links

Followed through using Autodesk structural designer.

Additional tensile force generated by the links reduces the moment capacity.

Longitudinal steel (2 layers ⌀25 at 150) is not sufficient

Repeated iteration using 2 layers ⌀32. Steel is sufficient in this case.

Centroid of pier

Part	dimensions (m) x	y	z	offset x (m)	c_x (m)	volume (m3)	force (kN)
Base	22.5	25	2	0	11.25	1125	28125
Front wall	1.5	21	10.5	0.5	1.25	331	8269
Back wall	1.5	21	13.8	20.5	21.25	435	10868
Side walls	21.5	3	13.8	0.5	11.25	890	22253
Pier walls	3	3	13.8	-	6.5	124	3105
Haunch	0.5	21	1.5	20	19.83	8	197
Overhang	12	24	1.2	10	16	346	8640
Cylinder base	3	4	3.5	2	3.5	42	1050
				cx =	11.81	3300	82506

Offsets and c calculated from front corner

eccentricity e_x = 0.56 m to back

5.2 Global pier check

Length of wall = 24 m

length of base = 25 m

	Individual loadcases	Unfactored moment (kNm)	ψ1	Factors (*UK NA to BS EN 1990*) EQU (A)	STR/GEO (B)	STR/GEO (C)	Factored FZ (kN) EQU (A)	STR/GEO (B)	STR/GEO (C)	Factored MY (kNm) EQU (A)	STR/GEO (B)	STR/GEO (C)
	Destabilising actions (from FX, FZ)	Reaction (kN)		Factors								
	Vertical reaction at deck support											
	Lever arm deck support to centre	6.5	m (TBC)									
			ψ1	EQU (A)	STR/GEO (B)	STR/GEO (C)	Factored FZ			Factored moments		
	DL+SDL	8461.5	-	1.05	1.2	1	8885	10154	8462	57750	66000	55000
Either	Gr1a + pedestrian x0.6	11076	1	1.35	1.35	1.15	14953	14953	12737	97192	97192	82793
or	Gr5	9753	1	1.35	1.35	1.15	13167	13167	11216	85583	85583	72904
	Wind (vertical)	972.5	0.2	1.7	1.7	1.45	331	331	282	2149	2149	1833
	TOTAL	20510					24168	25437	21481	157091	165341	139626
	Longitudinal reaction at deck support											
	Lever arm (Base to deck support)	14.50	m									
			ψ1	EQU (A)	STR/GEO (B)	STR/GEO (C)	N/A			Factored moments		
	LL (longitudinal)	900	1	1.35	1.35	1.15				17617.50	17617.50	15007.50
	Wind (longitudinal)	358.9	0.2	1.7	1.7	1.45				1769.38	1769	1509
	TOTAL	1258.9								19387	19387	16517
	Self weight of concrete											
	Eccentricity of concrete SW	0.56	m									
			ψ1	EQU (A)	STR/GEO (B)	STR/GEO (C)	Factored FZ			Factored moments		
	Self weight	82506	-	0.95	0.95	1	78380	78380	82506	43540	43540	45831
	TOTAL	82506					78380	78380	82506	43540	43540	45831
	Destabilising actions (Direct moments)											
	Overturning earth pressure	65912	-	1.05	1.35	1	N/A			69208	88981	65912
	Overturning surcharge	11172	-	1.35	1.35	1				15083	15083	11172
	Overturning cantilever moment:											
	wind (assumed non leading variable)	12257	0.2	1.7	1.7	1.45				4167	4167	3555
	Gr1a	67389	-	1.35	1.35	1.15				90975	90975	77497
	Gr5	61124	-	1.35	1.35	1.15				82517	82517	70293
	Longitudinal loads	900	-	1.35	1.35	1.15				2005	1215	1035
	DL	104002	-	1.05	1.2	1				109202	124802	104002
Either	Gr1a+longit+DL+EP+Surch+Wind vertical	253543								290640	325224	263173
or	Gr5+DL+EP+Surch+Wind vertical	246378								280177	315551	254934
	Total Destabilising	253543					102548	103817	103987	467118	509952	419316
	Stabilising actions											
	Eccentricity of concrete SW	45831		0.95	0.95	1	43540	43540	45831	43540	43540	45831
	Stabilising water pressure (low tide)	4905		0.95	0.95	1	4660	4660	4905	4660	4660	4905
	Total stabilising	4905					4660	4660	4905	48199	48199	50736
	Total	**248,638**		**-**	**-**	**-**	**97,888**	**99,158**	**99,082**	**418,918**	**461,752**	**368,580**

5.3 Pile design

Pile numbers

Assume that the moment is resisted by a piled foundation on a pile cap with demensions L (trans) 25 m

B (longit.)= 22.5 m

Assume pile spacing (centre-centre) = *each way* 2.9 m longitudinal

3.2 m transverse

Number of piles therefore = *transverse* 8.00

longitdinal 8.00

Assume even split between the rows

Distribution within one column of piles:

Results on piles (SET B)

ΣMY = 57,719 kNm per pile column

ΣFZ = 103817 kN total

Pile	y	FZ_{MY}	FZ_{FZ}	Σ(FZ)			
1	10.15	1658.6	1622.147	3280.7	Max =	3280.7	kN
2	7.25	1184.7	1622.147	2806.9	Min =	-36.4	kN
3	4.35	710.8	1622.147	2333.0			
4	1.45	236.9	1622.147	1859.1			
5	-1.45	-236.9	1622.147	1385.2			
6	-4.35	-710.8	1622.147	911.3			
7	-7.25	-1184.7	1622.147	437.4			
8	-10.15	-1658.6	1622.147	-36.4			

Results on piles (SET C)

ΣMY = 46,072 kNm per pile column

ΣFZ = 103986.55 kN total

Pile	y	FZ_{MY}	FZ_{FZ}	Σ(FZ)			
1	10.15	1323.9	1624.79	2948.7	Max =	2948.7	kN
2	7.25	945.7	1624.79	2570.4	Min =	300.9	kN
3	4.35	567.4	1624.79	2192.2			
4	1.45	189.1	1624.79	1813.9			
5	-1.45	-189.1	1624.79	1435.7			
6	-4.35	-567.4	1624.79	1057.4			
7	-7.25	-945.7	1624.79	679.1			
8	-10.15	-1323.9	1624.79	300.9			

APPENDIX A.2. – EXAMPLES OF MOVABLE BRIDGES

Type of Bascule Bridge		Label	Photo	Year of Construction	Location
Bascule Rolling Bridge	Scherzer (Half-through single leaf)	Pegasus	 (Wikipedia, n.d.)	1934	Bénouville, France
	Rall	Chicago and Illinois Western Railway Bridge	 (Holth, n.d.)	1914	Cook County, Illinois

Simple Trunnion or 'Chicago'		Morrison Bridge	 (VanderHart, 2015)	1958	Portland, Oregon
Strauss or Multiple Trunnion	Heel-Trunnion	Johnson Street	 (Cacophony, 2004)	1924	Victoria, British Columbia

Strauss or Multiple Trunnion	Overhead Counterweight	Green Bank Road	 (Birnstiel, Bowden, & Foerster, 2015)	1993	Burlington County, New Jersey
	Underneath Counterweight	Burnside Bridge	 (Morgan, 2012)	1926	Portland, Oregon

Modern Belidor (with hydralic mechanism)	Leça Movable Brige		2007	Leça da Palmeira (Portugal)

Type of Swing Bridge	Label	Photo	Year of Construction	Location
Centre bearing	Third Avenue	 (Wikipedia, n.d.)	1898	Manhattan New York City
Rim bearing	Glebe Island	 (Mitchell, 2006)	1903	Rozelle (Australia)

Cable-stayed	Media City Footbridge		2011	Salford Quays (United Kingdom)

Type of Vertical Lift Bridge	Label	Photo	Year of Construction	Location
Tower drive	Millennium Lift		2000	Salford Quays (United Kingdom)
Span drive	Path Hackensack River Bridge		1928	Kearny and Jersey City, New Jersey

(Holth, HistoricBridges.org Features, 2013)

Connected tower drive	Lovelandtown Bridge	 (Blog at WordPress.com., 2014)	1972	New Jersey Hammonton